

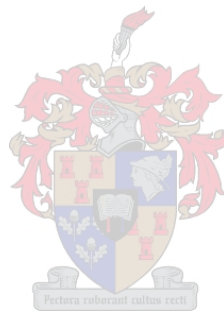


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Reliability of cold-formed steel screwed connections in tilt-and bearing

by

Rudolf van Wyk



Thesis presented in fulfilment of the requirements for the degree of

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Supervisor:

Dr. Celeste Viljoen

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Declaration

By submitting this thesis electronically, I declare that the entirety of the work contained therein is my own, original work, that I am the authorship owner thereof (unless to the extent explicitly otherwise stated) and that I have not previously in its entirety or in part submitted it for obtaining any qualification.

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Abstract

The South African National Standard for the structural use of cold-formed steel (SANS 10162-2) provides capacity prediction models for screwed connections. Screwed connections are designed against shear failure of the screw(s), section tear-out, net section failure and tilt-and-bearing failure. Previous studies (Rogers & Hancock, 1997) showed that the capacity is typically determined by the tilt-and-bearing type failure mode. The aim of this document is to report on the reliability of single screwed connections in cold-formed steel against this critical failure mode.

Predicted nominal capacities depend on the ultimate tensile strength of the steel, the thickness of the connected plates and the diameter of the screw. Design capacities are obtained by multiplying the nominal capacities by a capacity reduction factor of 0.5, according to SANS 10162-2. Reliability is assessed by means of FORM analyses, taking uncertainty in the prediction model and variability of input parameters into account.

Laboratory testing of 222 single screwed connections allowed to statistically describe the model factor, i.e. the ratio of actual tested- over unbiased predicted capacity. For each connection, the steel strength, plate thickness and screw diameter were measured, with the measured values used to predict capacity. This implies that the model factor accounts for uncertainty in the prediction model and experimental setup, while the variability of input parameters is separately accounted for through appropriate statistical modelling.

Variability in the input parameters was described using appropriate statistical distributions from expert literature (Holicky, 2009:199; JCSS, 2000). For steel strength, the mean value and standard deviation were obtained from tensile tests, while mean values and standard deviations of the plate thickness and screw diameter were obtained from the above mentioned measurements.

The experimental work and numerical analysis resulted in a model factor with a mean just exceeding unity and a small standard deviation. This suggests that the design code under consideration is able to accurately predict the nominal capacity of screwed connections. The FORM analysis resulted in computed reliability indexes significantly higher than the corresponding target values which suggest conservative and reliable design formulations.

Samevatting

Die Suid-Afrikaanse Nasionale Standaard vir die strukturele gebruik van koud gevormde staal (SANS 10162-2) bied voorspellingsmodelle vir die kapasiteit van skroef verbindings. Skroef verbindings word ontwerp teen skroef faling, staal profiel faling, die uitskeer van skroewe en ook faling weens skroef kanteling. Vorige studies (Rogers & Hancock, 1997) het getoon dat die kapasiteit gewoonlik bepaal word deur die skroef-kantel falingsmodus. Die doel van hierdie navorsing is om verslag te doen oor die betroubaarheid van tipiese enkel skroef verbindings in koud gevormde staal strukture teen hierdie kritiese falingsmodus.

Voorspelde nominale kapasiteite hang af van die treksterkte van die staal, die dikte van die verbonde profile en die diameter van die skroef. Volgens die SANS 10162-2 word die ontwerp kapasiteit verkry deur die nominale kapasiteit met 'n kapasiteitsverminderingfaktor van 0.5 te vermenigvuldig. Betroubaarheid word ontleed deur middel van 'n eerste orde betroubaarheidsmetode analise, met die in ag neming van onsekerheid in die voorspellingsmodel en wisselvalligheid van die parameters.

Laboratoriumtoetse van 222 enkel skroef verbindings het 'n statistiese beskrywing van die model faktor toegelaat. Die model faktor is bereken as die verhouding tussen die getoetste kapasiteit en die voorspelde kapasiteit. Die staal sterkte, profiel dikte en skroef diameter is gemeet vir elke verbinding met die gemete waardes wat gebruik is om die kapasiteit te voorspel. Dit beteken dat die model faktor slegs onsekerhede in die voorspellingsmodel en van die eksperimentele opstelling in ag neem, terwyl die wisselvalligheid van die parameters afsonderlik in ag geneem word deur toepaslike statistiese modellering.

Variasie in die parameters is beskryf met gepaste statistiese verdelings voorgestel deur verskeie literatuur (Holicky, 2009:199; JCSS, 2000). Aangaande die staal sterkte, is die gemiddelde waardes en standaardafwykings verkry deur standaard trek toetse terwyl die gemiddelde waardes en standaardafwykings van die plaat dikte en skroef diameter verkry is deur die bogenoemde metings.

Die eksperimentele werk en numeriese analise het gelei tot 'n model faktor met 'n gemiddeld hoër as een en 'n klein standaardafwyking. Dit dui daarop aan dat die ontwerp-kode onder oorweging in staat is om die nominale kapasiteit van skroef verbindings akkuraat te voorspel. Die betroubaarheid analise het gelei tot betroubaarheidsindekse aansienlik hoër as die ooreenstemmende teiken waardes wat daarop dui dat die ontwerp formulerings betroubaar en hoogs konserwatief is.

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List of Abbreviations

AISI	American Iron and Steel Institute
ALS	Accidental Limit State
AS	Australian Standards
ASTM	American Society for Testing and Materials
AZ	Aluminium-Zinc
BMT	Base Metal Thickness
CAD	Computer Aided Design
CFS	Cold-Formed Steel
CoV	Coefficient of Variation
DSM	Direct Strength Method
EC	Euro Code
FLS	Fatigue Limit State
FORM	First Order Reliability Method
GoF	Goodness of Fit
HRS	Hot-Rolled Steel
ISQ	International System of Quantities
LN	Log-Normal Distribution
LRFD	Load and Resistance Factor Design
LSF	Limit State Function
LSFB	Light Steel Frame Building
NAS	North American Specifications
NZS	New Zealand Standards
RC	Reliability Class
SABS	South African Bureau of Standards
SANS	South African National Standard
SASFA	South African Steel Frame Association
SLS	Serviceability Limit State
TCT	Total Coated Thickness
ULS	Ultimate Limit State
VaP	Variable Processor

List of Symbols

A_n	Nominal cross sectional area
a_d	Dimensions design value
β	Reliability index
β_t	Target reliability index
b_o	Specimen original width
C	Bearing factor
d_{sh}	Fastener shank diameter
d_f	Nominal fastener diameter
E_d	Design action effect
e	End distance
f_d	Material properties design value
f_u	Ultimate tensile stress
f_y	Yield stress
$f_{y,\mu}$	Mean yield stress
$f_{y,k}$	Characteristic yield stress
$f_{ult,\mu}$	Mean tensile stress
$f_{ult,k}$	Characteristic tensile stress
G550	Grade 550 MPa steel
$g(X)$	Limit state function
κ	Kappa
l_a	Connection specimen lap length
$LN(\mu, \sigma)$	Log-normal distribution
L_o	Specimen original gauge length
L_c	Specimen original parallel length
$N(\mu, \sigma)$	Normal distribution
N_f	Net section capacity
P	Probability
P_f	Probability of failure
P_t	Target level of probability
R	Resistance effect
R_d	Design resistance effect
R_{model}	Predicted resistance
R_{test}	Tested resistance
s_f	Section width

S_o	Specimen original cross section
t	Thickness
U	Standardised random variable (X)
u	Realisation of random variable U
u_i^*	Standardised design point coordinate
V_b	Bearing capacity
V_f	End tear out capacity
w	Coefficient of variation
X	Random variable X
x	Realisation of random variable X
x_{mod}	Random variable X modulus
χ^2	Chi-square test statistic
$\chi_{\alpha,v}^2$	Chi-square percentage point
Y	Random variable Y
y	Realisation of random variable Y
Z	Standardised random variable
$^{\circ}\text{C}$	Degrees Celsius
ϕ	Capacity reduction factor
$\Phi(x)$	Distribution function
$\varphi(x)$	Probability density function
θ	Model factor
θ_d	Model factor design value
μ	Mean
μ_{xi}^e	Mean of random variable X_i
σ^2	Variance
σ	Standard deviation
ρ	Correlation coefficient
α	Sensitivity factor
ε	Kurtosis

Chapter 1

Introduction

1.1 Structural use of Cold-formed steel

The South African (SA) housing industry has, for many years, primarily been dealing with masonry and timber as the go-to building materials for wall- and roof construction respectively. In 2006, light, thin and economical Cold-Formed Steel (CFS) was formally introduced to the SA market as an alternative building material. Although CFS profiles have been available for many years before 2006 it was never really used as primary structural elements in the South African engineering practice. CFS are used for more than the framing of structures, but in the context of this thesis the term CFS refers to the steel used in Light Steel Framed Structures (LSFS). Started off being only used for low-cost single story housing, the light steel frame building industry has grown and are now being used for multi-storey commercial, office and industrial buildings. Although it is seen as fairly new in SA, CFS has been used from as early as 1850 in countries like the United States and Great Britain (Allen, 2006).

The use of CFS was limited to a few basic structures as no adequate design standard for CFS structures existed. The first specification for the design of light steel frame building was published in 1940 by the American Iron and Steel Institute (AISI). Since then it was regularly updated with the 2011/2012 edition being the most recent. It was in the 1960's that Australia joined in on this alternative type of building material and in 1986 that the first Australian standard (AS 4100) was published. After teaming up with Standards New Zealand, they published the AS/NZS 4600:1996. It was the latest version of the AS/NZS 4600:2005 that was adopted as SANS 10162-2, the South African National Standard for the structural use of CFS (Hancock, 2007).



1.2 Reliability Analysis

In any aspect of structural design, problems must be resolved in the face of various uncertainties. A wide range of uncertainties is present through the design stage and during construction. The main sources of uncertainties can be attributed to human error, incomplete knowledge, natural randomness of materials and simplified mechanical formulations used to predict and describe the response and capacity of the structure. Past generations have used design methods that failed to accurately and rationally assess these uncertainties and therefore an alternative approach was necessary. This came in the form of structural reliability techniques that are able to rationally quantify and assess the effects of uncertainties. The use of these techniques involves the representation of the uncertainties with the use of probability distributions and a probabilistic assessment of the structural performance. By using distributions, the variability in structural properties and performance are taken into account (Mensah, 2012).

Uncertainties regarding the natural randomness of materials are well understood whereas those directly associated with capacity prediction models are often not as well described. Prediction models are usually expressed as a complex function of various material and geometric parameters. The variability (uncertainty) of these parameters can be taken into account through probabilistic modelling. This probabilistic analysis is an effective way of incorporating the uncertainties associated with the material and geometrical parameters used in prediction models.

When a prediction is made using the true (measured) values of the input model parameters and the model predicted value deviates from the true (tested) value it is an indication of uncertainties present within the model itself and also the test set-up. These uncertainties are statistically assessed by using a model factor which is defined as the tested value over the unbiased predicted value. The model factor mean indicates whether the prediction model, on average, is conservative or unconservative where its variation (standard deviation) indicates the level of uncertainty associated with the model.

The reliability of a structure or structural component is determined through a reliability analysis. It is routine to separate the reliability assessment of loads and resistance, by recognising that each contribute in part to the total reliability. During analysis of resistance reliability all uncertainties must be taken into account, including material-, geometric- and model uncertainty. As part of reliability analyses, reliability methods are used as fundamental procedures. An example of one of the basic and very efficient reliability methods is the First



Order Reliability Method (FORM) which delivers a reliability index that must meet the specified target reliability index in order for the structure to be declared reliable (Holicky, 2009).

1.3 Problem statement

Since the formal introduction of CFS in LSFS in South Africa, it has become more popular and due to the increase in the structural use of CFS, the need for a new national design standard to replace the outdated first edition also increased. This need was satisfied in the form of SANS 10162-2:2011 which is the second and latest design standard for the structural use of CFS in South Africa. By applying some national modifications regarding the material and geometrical properties of CFS members, the SANS 10162-2 was adopted from the Australian and New Zealand CFS design standard (AS/NZS 4600: 2005). The fact that the SANS 10162-2 is a modified adoption may raise some concern on its applicability and reliability in SA. A previous study (Oosthuizen, 2010) investigated the reliability associated to element design of a typical building to SANS 10162-2, but the reliability of connections have yet to be determined. This thesis investigated the reliability associated with screwed connections expected to fail in screw tilting and hole bearing.

Various previous related studies (Section 2.8) have identified some highly conservative design equations, some unconservative equations and even rationally incorrect formulations for both screwed and bolted connections subjected to shear. Although it is only screwed connections considered here, inaccurate models of any kind give reason to doubt the accuracy and reliability of the SANS 10162-2 as a whole. The problem statement can thus be concluded by saying that the reliability of screwed connections designed according to SANS 10162-2 is questionable. The purpose of this thesis is to report on the reliability of the SANS 10162-2 regarding screwed connections expected to fail in tilting-and bearing.

1.4 Project Objectives

The SANS 10162-2 provides capacity prediction models for screwed connections which are designed against shear failure of the screw(s), section tear-out, net section failure and tilt-and-bearing failure. Previous studies (Rogers & Hancock, 1997) showed that capacity is typically determined by the tilt-and-bearing type failure mode. The aim of this thesis is to report on the reliability of single screwed connections in CFS against this critical failure mode.



1.4.1 Model factor

The best way to investigate and report on the ability of the capacity prediction model, for screwed connections, to accurately predict the true resistance capacity is to compare the predicted values obtained from using the design standard to collected data from experimental work. When the unbiased predicted resistance capacity deviates from the true capacity, it is an indication of the level of uncertainties present. The random variability of the input parameters was incorporated through the probabilistic modelling of the respective parameters, where the uncertainties directly associated with the prediction model were quantified in the form of a probabilistic model factor. The determination and statistical analysis of such a factor for the prediction model of CFS screwed connections in tilt-and bearing was the first of two objectives of this research. The ratio of the tested and predicted capacities as obtained through experimental work and unbiased capacity predictions was used as an indication of how accurately the resistance prediction model predicted the true (tested) connection capacity.

1.4.2 Reliability analysis

In order to report on the reliability of the SANS 10162-2 with regards to screwed connections expected to fail in tilting and bearing, reliability analyses were performed. A reliability analysis aims to rationally quantify and assess all uncertainties associated with the model and materials under consideration. Through a rational quantification of all identifiable uncertainties, the second objective of this research was to determine the reliability status of typical screw connections in tilt-and bearing. Through the use of FORM, the reliability analysis delivered a reliability index associated with the resistance prediction model. In order for the connection design to be declared sufficiently reliable, this computed reliability index has to meet a specified target index value.

1.5 Structure of thesis

This section summarizes the outline of the thesis. Brief descriptions of each of the chapters are given, after which a graphical illustration of the thesis outline is given in Figure 1.1.

- Chapter 1: Introduction (current chapter).
- Chapter 2: Literature regarding CFS in general and the structural use of CFS are discussed in this chapter. The history and necessary background information regarding CFS design standards are also described. In the form of previous relevant studies, this chapter provides a fair amount of evidence in support of the current investigation.



- Chapter 3: Literature regarding structural reliability is discussed in this chapter. This chapter provides a good basis for the reliability analyses performed in Chapter 7.
- Chapter 4: Chapters 1-3 have discussed the necessary literature that serve as a good background for the experiments and analyses that follow in Chapters 5-8. The research methodology regarding the experiments and analyses is introduced in this chapter.
- Chapter 5: This chapter discusses the experimental design of the project as well as the experimental results. Two experiments were designed and executed. Tensile tests were performed in order to obtain the tensile strength of the steel and connections were tested to obtain connection capacities.
- Chapter 6: Using the results of Chapter 5, this chapter discusses the model factor determination.
- Chapter 7: By using Chapter 3 as background, this chapter discusses the reliability analyses performed using the experimental results and various predictions. The results of the analysis are also discussed here.
- Chapter 8: This chapter ends the thesis with conclusions and recommendations for further studies.
- Appendices: Two appendices are attached at the back of thesis. Appendix A gives the complete set of experimental results where the statistical parameters of the results are given in Appendix B.

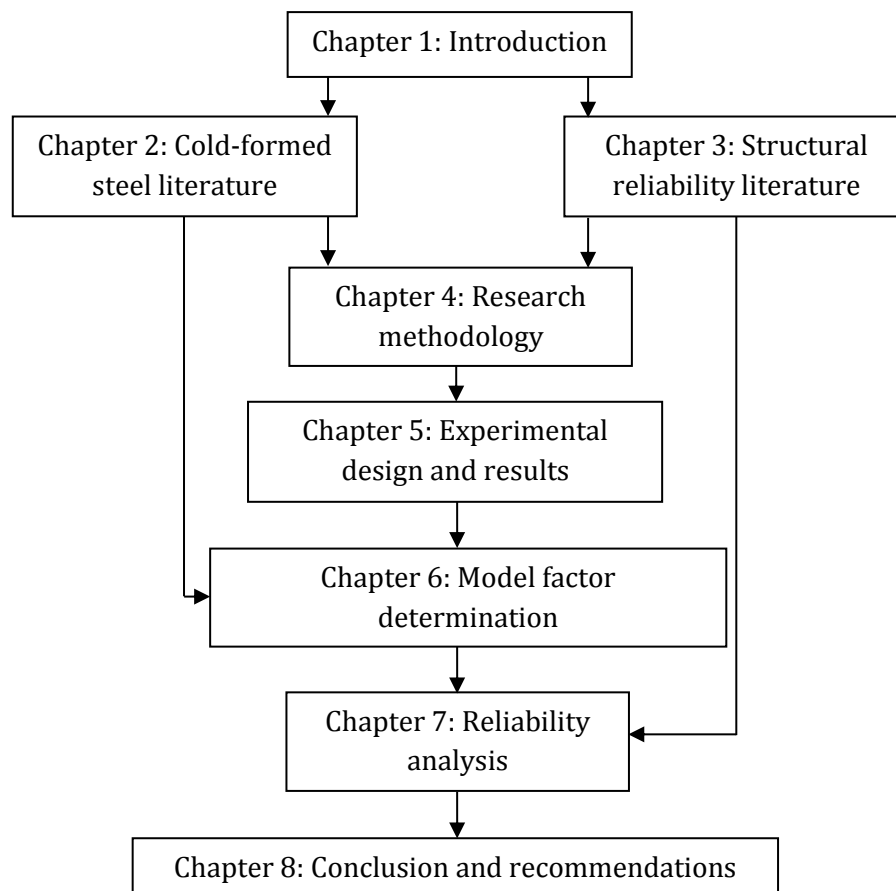


Figure 1.1 Thesis chapter outline

Chapter 2

Cold-formed steel

2.1 Introduction

For many years the first thing that came to mind when mentioning steel construction is large heavy Hot Rolled Steel (HRS) members with unfinished surfaces used as beams and columns. This has however changed with the introduction of lighter and thinner steel, known as Cold-Formed Steel (CFS). Made from high strength galvanised steel sheets, CFS is shaped and formed at ambient temperatures with the use of computer controlled mills. Although it is used for a wide range of applications, CFS is primarily used for structural purposes. The structural use of CFS is often referred to as Light Steel Frame Building (LSFB). Ranging from small residential buildings to multi-storey office buildings, the properties of CFS allows for complex and inventive structural designs. Members can be connected through a number of fastening systems of which bolting and screwing is the most commonly used. This chapter reviews the necessary literature to provide a clear understanding of the different aspects of CFS, the design considerations and lightweight steel construction.

2.2 Background

2.2.1 Cold-formed steel

Although many see CFS as a new construction material, it has been used from as early as 1850 for residential purposes in countries like Great Britain and the United States. Since the introduction of CFS in 1850 until the early 1900's, the use of CFS was largely experimental and limited to a few basic structures, because of no adequate design standard for CFS structures. The first real construction using CFS was done in 1925 where it was used to support the floor of a newly build Baptist hospital in Virginia, USA. It was in 1933 that architect Howard T. Fisher developed a house completely framed with CFS. The main attributes of this house that gave it



the “home of the future” status was the relatively low price and the short construction time of only four days (Dubina et.al, 2012; Allen, 2006).

During and after World War II the lightweight steel framing industry began to grow and was primarily used for residential purposes and other facilities that required rapid, lightweight and string construction. In the 1960's Australia joined in on this alternative type of building construction and in the late 1980's they led the world in house framing, using 550 MPa grade CFS. Already being used for decades in Australia, USA and Britain it was only in the late 1990's that South Africa started using this light steel for roof trusses and wall frames (Du Preez, 2012; Eticon Construction, 2012).

According to the director of the Southern African Light Steel Frame Building Association (SASFA), John Barnard, it was in 1999 that light steel frame roof trusses started being supplied into the South African market. In South Africa, timber has always been the first choice when it came to building roof trusses, but due to an increase in timber prices, a shortage in good quality structural timber and an increase in the awareness of the advantageous use of CFS, the use of light steel trusses have been growing exponentially. In 2006 the light steel frame industry was formally introduced in South Africa, which meant that the CFS was now being used for more than just roof trusses. Due to South Africa's limited history of LSFB, the growth of CFS construction started off slowly. LSFB were initially only used for single storey residential projects, but are now being used for multi-storey commercial, office and industrial buildings (Barnard, 2011).

Although it is often compared, CFS is not seen as a replacement or an alternative to HRS, but rather as another type of steel. However, when being compared to masonry as wall components and timber as roof trusses, CFS is seen as an adequate alternative building material. Due to its low mass, ease of application and a vast amount of advantages, there are more than enough reason to believe that CFS will sooner rather than later become one of the more popular building materials in South Africa.

2.2.2 Design standards

Most of the early development regarding steel construction took place in America and in 1855, the American Iron and Steel Institute (AISI) was founded. Although requirements for HRS had already been adopted into building codes in the 1930's, no such requirements were developed for CFS. This largely limited the use of CFS. Due to the difference in structural behaviour between HRS and CFS it was seen as impractical to apply HRS design standards to CFS structures (Allen, 2006).



The first specification for the design of light gage steel structural members was published by the AISI, already in the 1940's. This design clause was based on research work done by Professor George Winter at Cornell University (Dubina et.al, 2012). Since then, it was regularly updated and newer versions were published with the 2011/2012 edition being the most recent. Based on the 1968 edition of the American Specifications, the first Australian standard (AS 1538) was published in 1974. In 1988 the 1974 edition was revised using the 1980 and 1986 editions of the AISI (Allen, 2006; Hancock, 2007).

Up until 1991 both the Australian and American standards were in permissible stress format. In 1991, the AISI produced a limit states version of their 1986 specification called the load and resistance factor design (LRFD) specification. These limit state and permissible stress design methods are discussed in Chapter 3. As the Australian standards are based on the American standards, all structural design standards in Australia were progressively converted to the limit state philosophy (Hancock, 2007).

Standards Australia published their first limit states design standards (AS 4100) in 1990. In 1992 the AS 4100 was used by Standards New Zealand to produce their first limit state design standard for CFS structures (NZS 3404). In 1996 a joint standard was prepared by Standards Australia/Standards New Zealand (AS/NZS 4600:1996). Based on the AISI 1996 this joint limit state design standard was common to both countries and made use of SI units (Hancock, 2007).

In 2001 the AISI specification was revised to the North American Specification (NAS) in order to include Canada and Mexico. This 2001 edition was then updated in 2004 to include the newly developed Direct Strength Method (DSM). Taking into account these additions in the NAS and recent research on CFS in Australia, the AS/NZS 4600:2005 is the latest edition of Standards Australia/New Zealand. The 2005 edition of the AS/NZS 4600 was used to develop the South-African National Standard (SANS) for the structural use of CFS structures. The SANS 10162-2:2011 was adopted from the AS/NZS 4600:2005 with various national modifications made (Hancock, 2001). Due to the increase in popularity of CFS in the building industry, South Africa had to develop a new loading code for light steel frame buildings. This came in the form of the SANS 517, which is now being used in conjunction with the SANS 10162-2 for light steel frame building (Hancock, 2007).



2.3 Advantages of CFS

“ –It is structurally sound, makes use of high strength galvanised steel sheet, generates minimal wastage of materials and is a quick building process, is energy-efficient through good insulation, is dimensionally accurate, and so on.”

– Sinethemba Gqibitole (ArcelorMittal)

In general, there is a long list of advantages, but as CFS is one of the newest and latest building materials, its advantageous use becomes clear when being compared with other building materials.

2.3.1 CFS vs. Timber

The larger portion of CFS sections used in the LSFB sector is used for roof trusses, replacing timber (Lip & Benoit, 2012). Although the materials differ completely, using CFS instead of timber still provides the same design flexibility and span capabilities. Other advantages that the use of CFS has over that of timber are listed below (Barnard, 2008).

- A CFS truss spanning 10 m weighs 30% less than that of a 10 m timber truss.
- Due to the lower mass, the transport costs are reduced.
- It will not distort, even when left out in the rain.
- Termites are not a threat when working with steel.
- More than 70% of steel scrap is recycled and then re-used in the production of new steel products.

2.3.2 CFS vs. Masonry

When it comes to the other parts of a structure, masonry is mostly used as building material. Some advantages that the use of CFS has over that of masonry are listed below (Clark, 2005).

- Walls with 90 degree corners are possible to erect when using CFS.
- The end cost, appearance and construction time of CFS structures can accurately be predicted before construction.
- CFS is resistant to cracking.
- CFS structures are easily modified, added to or demolished with minimal site impact.
- The use of CFS is more energy and environmentally efficient as it provides good insulation and generates minimal wastage of materials.
- Cheaper foundations due to lower wall mass.



2.3.3 Problems of CFS design

Due to the thinness of CFS sections and through the process of cold forming there are a number of design problems that are encountered. A list of the most common problems is given below (Dubina, 2012).

- Low buckling strength.
- Low torsional stiffness.
- Web crippling.
- Due to the effect of cold forming, CFS sections have a low ductility.
- Complex connection design.

2.4 Production of CFS

Shaped and formed at ambient temperature, CFS members originate from flat steel sheets. The properties of the steel sheets influence the member properties and are therefore first discussed followed by a discussion of how the members are made.

2.4.1 Steel sheets

Initially the steel sheets are rolled to size in a hot strip mill with finishing temperatures of 940°C and coiling temperatures of 670°C. The coil of steel is then uncoiled and cleaned in an acid solution to remove surface oxides. After this treatment, the steel is trimmed and fed into a cold reduction mill where high compression and strip tension systematically reduce the thickness until the desired thickness is reached. This milling process increases the material strength and decreases the material ductility due to distortions in the grain. This distortion however can be changed through subsequent heat treatment of which a wide variety exists (Rogers & Hancock, 1997).

In the case of 300 MPa grade steel, the steel is heated to such a degree that recrystallization takes place which restores the original grain. For 550 MPa grade steel, the steel is stress relieved annealed which happens when the steel is heated to just below the recrystallization temperature. The steel is held at this temperature until the temperature is constant throughout the thickness that is followed by a slow cooling process. To improve the finishing quality and the flatness of the coil, it is further processed through a tension levelling mill (Rogers & Hancock, 1997).



2.4.2 CFS structural members

Once the steel plates are produced and neatly finished off it is used to make CFS structural members through either roll forming or press braking. Roll forming is a continuous process that shapes flat sheets of steel into profiles or sections through a machine called a roll former. As the steel passes through the rolls of the roll former, the rolls turn in the opposite direction drawing the steel through the machine (see Figure 2.1). The number of roll stations used during production depends on the complexity of the required section as each pair of rolls slightly changes the shape and geometry of the sheet. The dimensions of the members are electronically transmitted to the profiling equipment that are controlled by computers and are able to cut and punch holes in the steel at precise locations specified by the user. Roll forming is mostly used to produce a large number of standard components in a short time. Press braking works at a much slower rate than roll forming and is rather used to produce a wide variety of components in smaller production runs. During press breaking the sections are manufactured by pressing the steel sheets, forming one or two bends at a time (Eticon Construction, 2012).

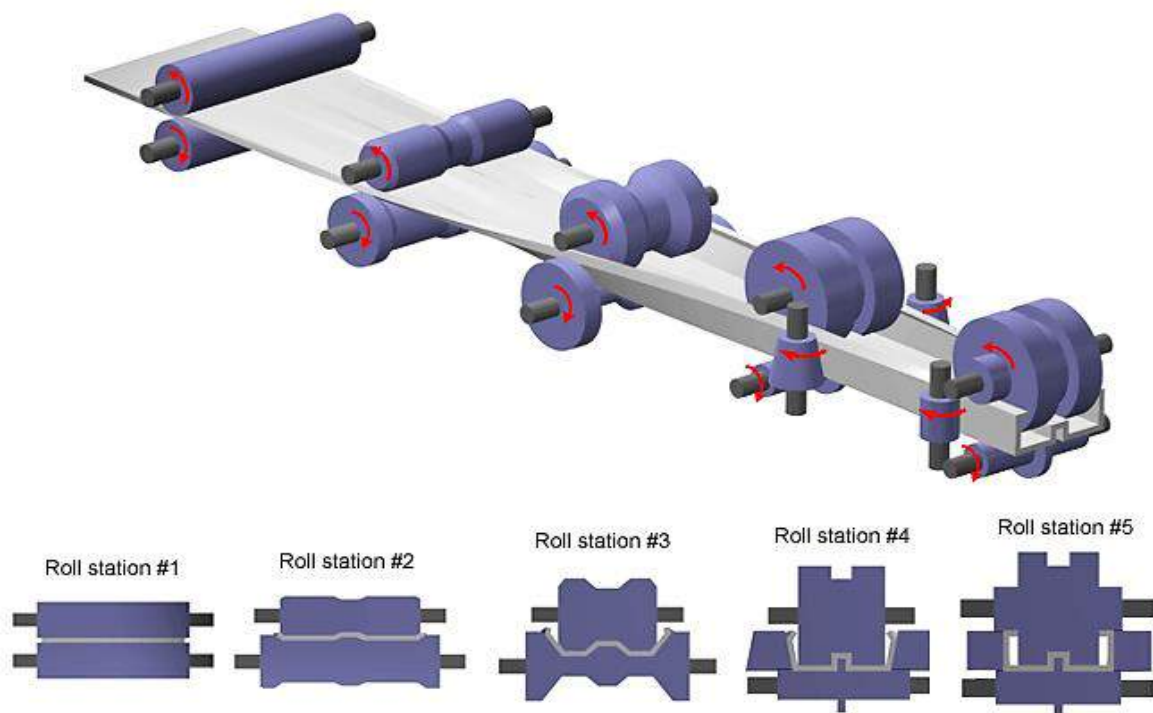


Figure 2.1 Roll forming process (Chicago Roll Company, 2014)



2.5 Light Steel Frame Building

2.5.1 Building components

CFS products are widely used in all aspects of modern life. Made from high strength galvanised steel sheets, CFS is used in automobiles, equipment, furniture and as building components just to name a few. When it comes to the use of CFS for building purposes, the components can be divided into two major types: Individual structural framing members and panels and decks.

CFS steel started off being used as secondary framing members, but are being used more and more as the primary framing of structures. CFS members are mainly used to carry loads and are available in a variety of shapes. Open sections are the preferred choice for building purposes, but CFS members are also available in closed and built-up sections. Examples of open sections are channels, lipped channels, hats, and angle sections while box sections and pipes are examples of closed sections. The built up sections are formed by connecting two or more steel members (AISI, 2010). Examples of these three sets of profiled sections are shown in Figure 2.2.

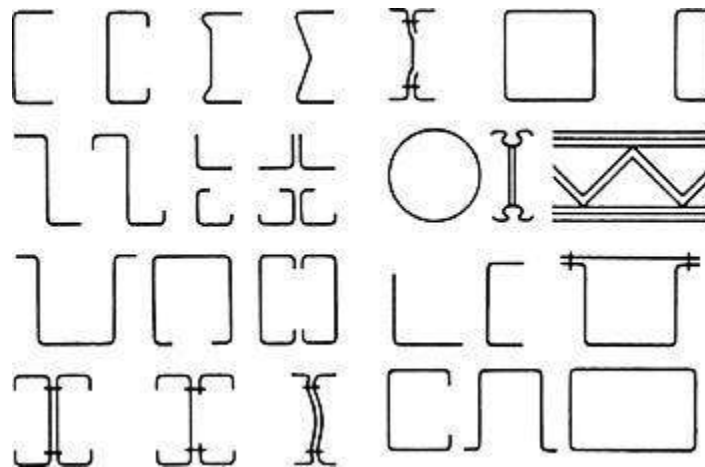


Figure 2.2 Typical CFS sections (Brockenbrough, 1999)

Closed, open or built-up members can be used as purlins, studs, trusses, braces and other building components to build light steel frame structures. A typical South African light steel frame building with the different building components are shown in Figure 2.3. Depending on the purpose of the steel member, wall thicknesses range from as little as 0.4 mm up to thicknesses of 6 mm with the depth of the sections ranging from 50 mm to 400 mm. Panels and decks are made from profiled sheets and cassettes with depths ranging from 20 mm to 200 mm and thicknesses ranging from 0.4 mm to 1.5 mm.

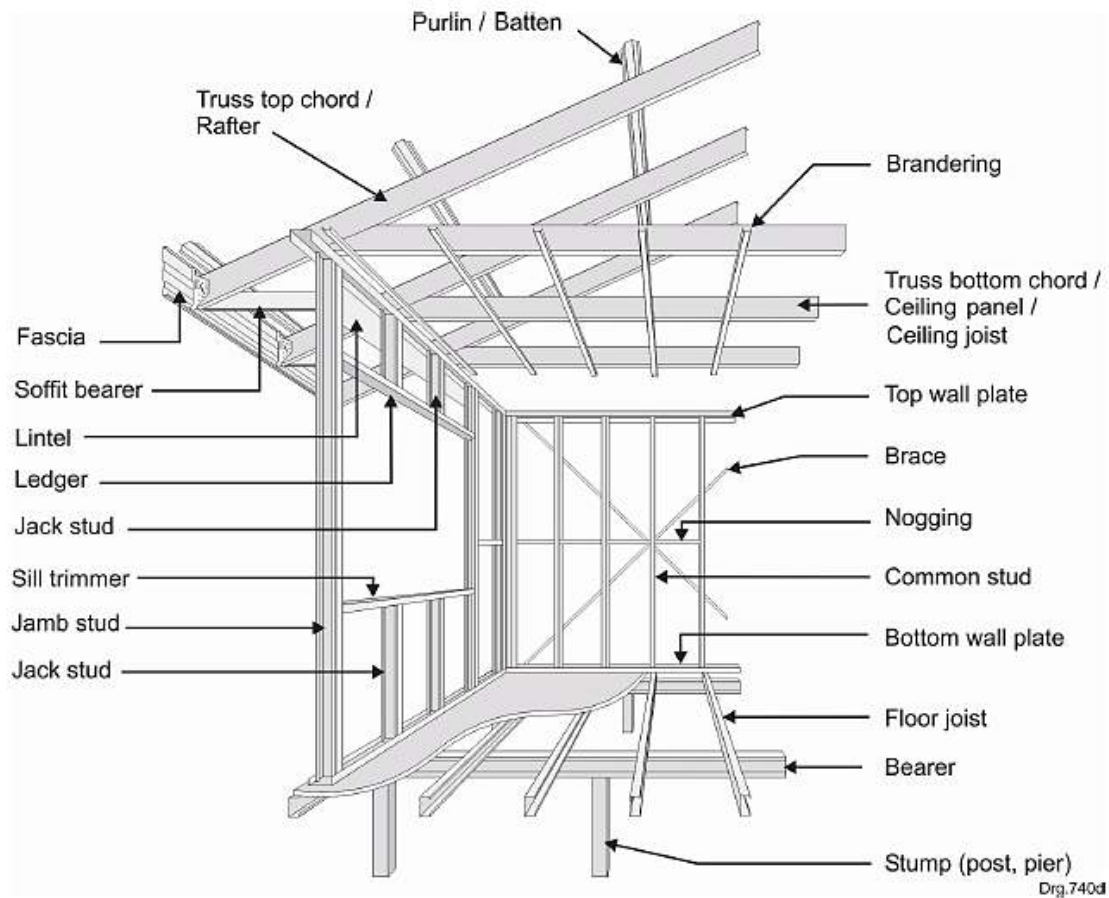


Figure 2.3 Typical light steel framed building (SABS, 2011)

2.5.2 Building process

When it comes to discussing the building process of light steel frame buildings, the beneficial use of CFS steel as a building material becomes clear. There are two basic approaches when it comes to constructing these buildings. The first approach is where standard CFS sections are available to builders who cut and assemble the structural components on site as they go along. The second approach is much more accurate and eliminates most of the possible human error. This method starts off by submitting a complete drawing of the structure to a framing company who designs the entire framing system and manufactures all of the structural components at the factory using computer software and specialised machinery. These components are given reference numbers to ensure that they are assembled according to the structural layout drawings. A team of factory workers then assemble the frame structure under factory conditions or to save logistic costs, it can be transported in bundles to site where the structure is assembled (Barnard, 2011).



The first building process mentioned is mostly used by private homebuilders where precise and very accurate dimensions are not of vital importance. However, for bigger and more specific structures, the second approach that makes use of computer software and machinery, is the preferred method as it promises to deliver highly accurate results (Barnard, 2011).

2.5.3 Buildings

Any completed structure can be subdivided into the roof, the walls and the floor. In light steel frame structures, CFS is used in all three of these subdivisions. Starting with the floor, the foundation plays a vital role in the design of the structure. A slight deviation in the squareness of the foundation can lead to a serious lack of fit in the framing of the structure as it is designed beforehand assuming square foundations. Regarding the construction of the floor itself, CFS is used as part of sub-floor structures and in composite steel concrete slabs in multi-storey frame buildings. This steel sub-floors usually consist of C-shaped cold-formed joists on top of either plywood or concrete, whichever can be installed to complete the floor (see Figure 2.4). Steel sub-floors are mainly used for light commercial structures like apartments and educational buildings (AISI, 2010; Dubina et.al, 2012).

Typically, the walls of a steel building consist of primary rigid frame members, secondary members, cladding and bracing. The primary rigid frames are fixed to one another using hand held power tools and the chosen fastening system. Primary frames can be seen as the main structure and the outline of the walls that are responsible for carrying and directing the applied loads. An example of such a framing system is shown in Figure 2.5. Once the frames are erected, it is anchored to the slab using the prescribed anchor bolts. The secondary members support the roof and the wall coverings and provide lateral stability to the primary rigid frame members. The bracing provides stability in the direction perpendicular to the primary frames with the cladding being responsible to transfer loads, such as wind and snow loads, to the secondary members (AISI, 2010; Dubina et.al, 2012).

Two of the major uses of CFS are the use of C sections as purlins and the construction of roof trusses. Roof trusses are fixed to the primary rigid frames using steel brackets and screws. Figure 2.6 shows an example of typical CFS roof trusses. The external wall frames are first covered with vapour permeable membrane to seal off the structure against air and wind ingress. Using special self-tapping screws the external cladding is fixed to the steel framed structure which is followed by the installation of roofing materials, providing a sheltered structure for the internal completion of the building. Before the wall cavities are filled with insulation and covered with the chosen panels, the plumbing and electrical work is done. Once



the panels are joined together the structure is ready for paint and other finishing touches (Barnard, 2008).



Figure 2.4 CFS used to support upper floors (SASFA, 2014)



Figure 2.5 Typical framing of CFS structures (SASFA, 2014)



Figure 2.6 Typical CFS roof truss (SASFA, 2014)

2.6 Connections

Connections can be defined as physical components used to fasten individual members in overall structures. Not only are connections responsible for structural stability, but is also important for transferring forces and moments from structural members to supporting elements. In CFS structures, connections are primarily used for interconnecting two or more members. Other than that it is used for connecting members to the supporting structure and for the assembling of bar members for light steel frame structures. CFS members can be connected in several ways and as connections play such a vital role in structures, it is important to choose the correct fastening method. According to the South African design standards for the structural use of CFS, the following categories can be used to classify connections found in LSF structures (Toma, 1993; Dubina, 2012)

- Screwed connections
- Bolted connections
- Riveted connections
- Welded connections

In order to select the most appropriate method of fastening for any given application, the following points need to be taken in consideration (Dubina, 2012).

- Load-bearing requirements
- Economic requirements
- Durability
- Water tightness
- Appearance



2.6.1 Types of connections

Bolts and screws are the most common fastener types used for CFS connections. Their ease of application and economy gives bolted and screwed connections its preferable status (Zadanfarrokh & Bryan, 1992).

2.6.1.1 Screws

Relatively low design loads and high level of redundancy make screw connections appropriate for bridging, lateral bracing and member to member connections. Figure 2.7 shows an example of screws used to connect multiple members. Due to the thinness of CFS, screw connections provide advantages in simple design and fast installation (Lee, 2014). Screws used to connect CFS members can be divided into self-tapping screws and self-drilling screws. Although it is sometimes not necessary to drill a small pilot hole, self-drilling screws form their own mating thread using a previously drilled pilot hole. The point of a self-drilling screw is tapered to a fine point with the thread of the screw running from the head to the point. These screws are mainly fabricated from heat-treated carbon steel and are mainly used to connect thin sheets to thin sheets.

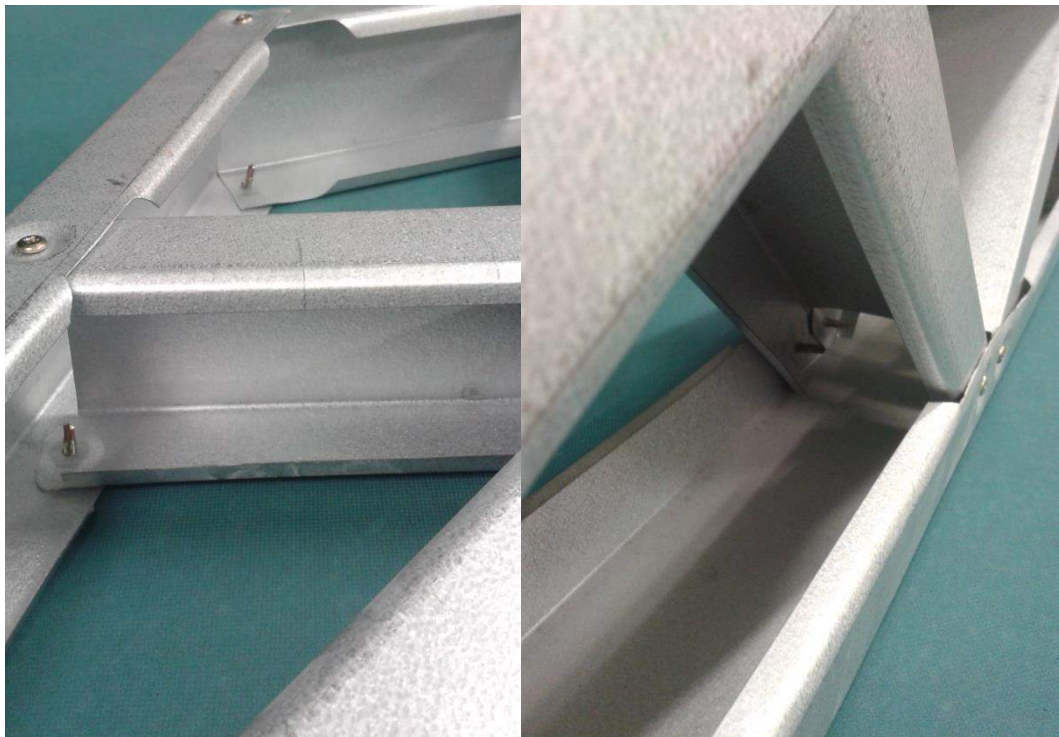


Figure 2.7 Typical use of screws in CFS structures

Self-tapping screws tap their own counter thread in a pilot hole prepared by the point of the same screw. The term “tapping” means to drill a pilot hole. Consisting of a small set of blades, the point of the self-tapping screw drills a small pilot hole after which the thread will pass



through the hole creating a thread in the material. Self-tapping screws can be divided into thread forming and thread cutting screws where thread forming screws is mostly used to connect thin sheets to thin sheets. Thread cutting screws are used to connect thicker steel sheets and as thread forming screws, it is fabricated from carbon steel.

In order to improve the bearing capacity and the sealing ability of screw connections, washers can be used in combination with screws. Elastomeric or combined metal elastomeric washers cause a reduction in the strength and stiffness of the connections. In order to improve resistance against corrosion, some of the screw types come with plastic heads or plastic caps (Dubina et.al, 2012).

Self-drilling and self-tapping screws each have a preferred material class with which it must be used in order to avoid the material to crack or a structure to fail. Self-tapping screws are the preferred choice when it comes to hard material like masonry and steel work. Figure 2.8 shows typical screws used for CFS connections.



Figure 2.8 Typical screws used for CFS connections

2.6.1.2 Bolts and nuts

Bolts are generally used for anchorage and bolting to thicker members with higher load demand. When using this type of mechanical fastening method, the holes through which the threaded bolts will fit are pre-formed in the factory which improves the quality and accuracy. Holes can be made as oversized, slotted or it can be made to have a perfect fit with the chosen bolt, preventing slipping. The bolts used for connection purposes of thin-walled sections need to be threaded close to the head and usually have diameters that range from 5mm to 16mm with the preferred property classes being 8.8 or 10.9 (Toma et.al, 1993).

Bolted connections can either be loaded in single shear (eccentric) or in double shear (concentric), with or without washers. Washers can be used under both the bolt head and the nut or only under either the bolt head or the nut. Bolt head shapes may be cup, countersunk, hexagonal or hexagonal flanged with the hexagon headed bolts being the most commonly used (Dubina, 2012). Typical bolts used for CFS connections are shown in Figure 2.10.



Figure 2.9 Bolts used for anchorage (Simpson Strong-Tie Company Inc., 2014)

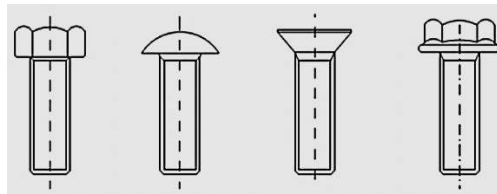


Figure 2.10 Typical bolts used in CFS construction

2.6.1.3 Blind rivets

Blind rivets, also known as pop rivets are often used in CFS construction. As for bolts, the holes are preformed when blind rivets are used. A pop rivet consists of a shank and a mandrel, with the mandrel passing through the middle of the shank. Using a pop rivet gun, the mandrel is pulled in the opposite direction which causes the blind end of the rivet to expand, ensuring a tight fit. Connections that are fastened with rivets generally fail in a similar manner as screwed connections (SABS, 2011).

2.6.1.4 Welds

The design procedures for welded connections in CFS structures are different than that of HRS. However, the welding itself is done using conventional equipment and electrodes used for hot rolled sections (Hancock, 2007). Welding of CFS sections can be done by arc welding as well as resistance welding (Toma et.al, 1993).

Welding uses a welding power supply to create an electric arc between an electrode and the work piece to melt the metals at the point to be welded together. During the welding process, the welding region is protected against harmful environmental conditions by some type of shielding gas (Toma et.al, 1993).



There is no electric arc involved when the resistance welding process is used. Instead, the welding current is led with high density from an electrode to the work piece. The work piece metal is heated to such an extent that it becomes plastic and then melts. The two sections are then locally connected as a result of a pressure transferred by the electrode to the metal work piece. Due to absence of an open electric arc, the protection discussed under arc welding is not necessary when performing resistance welding (Toma et.al, 1993).

2.6.2 Modes of failure

Depending on the type of fastening system used and whether it is subjected to a tensile, shear or compression force, connections in CFS structures have a variety of possible failure modes. When subjected to shear, bolted and screwed connections have the same expected failure modes. Both bolted and screwed connections, when subjected to shear, are expected to fail according to one of the following four failure modes.

2.6.2.1 End tear out

A connection will usually fail due to end tear out when the distance from the centre of the fastener to the end of the specimen is less than three times the nominal diameter of the fastener. This failure mode will start off with the sheet piling up in front of the fastener after which longitudinal shears will start to form at the piled up material and move towards the end of the specimen (Rogers, 2000) (See Figure 2.11).

2.6.2.2 Tilting and Bearing

When the end distance is not a problem, bolted and screwed connections tend to fail due to hole bearing and screw tilting. This failure mode can be pure tilting or pure bearing, but is often found to occur in combination. Bearing is most likely to be dominant when thick members are fastened using bolts where tilting is expected to happen for screwed connections of thinner members (Dubina, 2012). The combined failure mode of tilting and bearing will start off with the steel undergoing bearing followed by the tilting of the fastener. Bearing is known as the piling up of the steel behind the fastener (Figure 2.12). A typical screwed connection that undergoes tilting is shown in Figure 2.14 (Rogers, 2000).

2.6.2.3 Tension failure in net section

When the steel section is not able to resist the force, the member will start to crack perpendicular to the direction of the force. This is shown in Figure 2.13. Bearing is often present prior to net section failure. In such cases, the net section failure is identified by the necking of the steel in line with fastener (Rogers, 2000).



2.6.2.4 Fastener in shear

When the strength of the net section is higher than the strength of the fastener, the fastener will fail first. The fastener will fail in shear as shown in Figure 2.15. This is a sudden brittle failure mode. This failure mode does not occur often in CFS structures due to the thinness of the sheets.

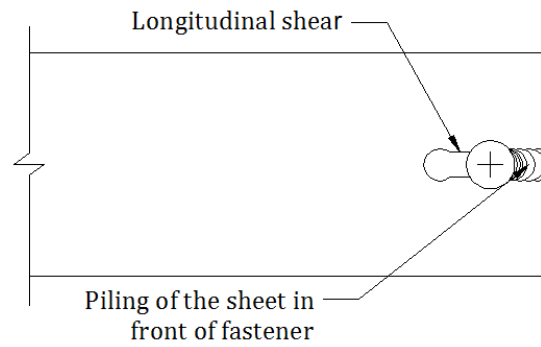


Figure 2.11 End tear out

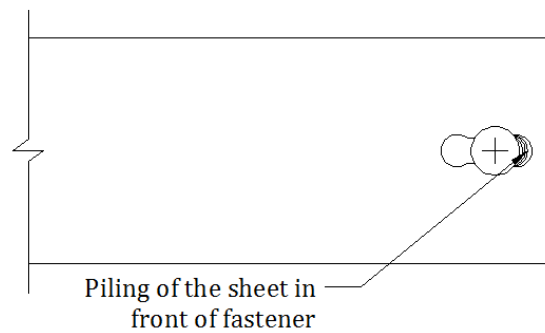


Figure 2.12 Bearing failure mode

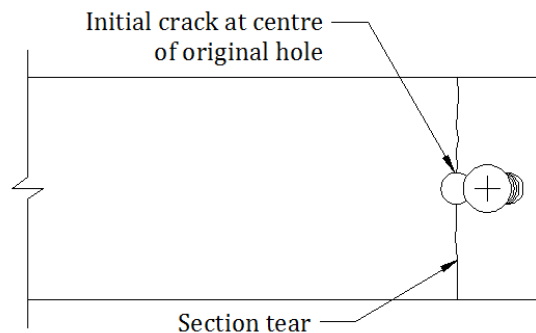


Figure 2.13 Net section tear

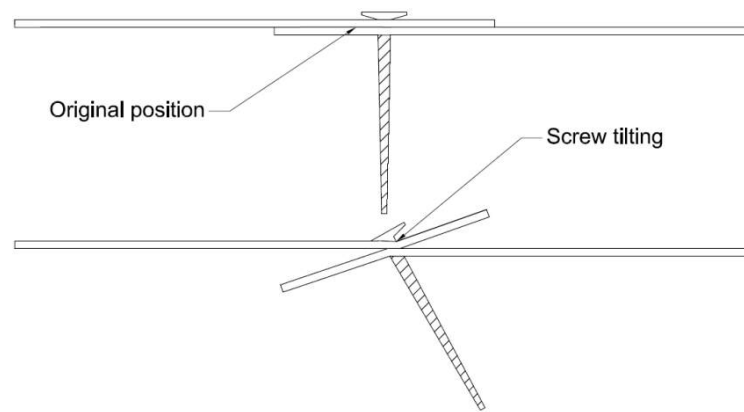


Figure 2.14 Tilting failure mode

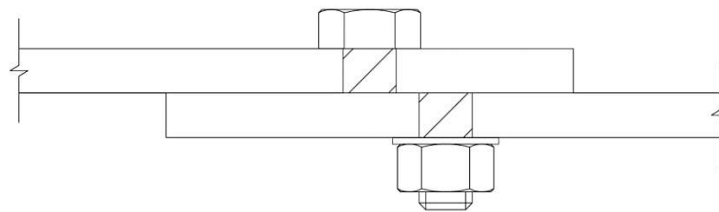


Figure 2.15 Shear of fastener

2.6.3 Connection properties

Before designing any sort of structural member or connection, it is important to be familiar with the properties of the steel sheets, plates and other elements generally used in CFS construction (Wei-Wen, 2000). Members in CFS construction are usually made from carbon structural steel or high-strength low-alloy structural steel. Any type or grade of steel derives its properties from a combination of chemical composition, heat treatment and manufacturing process (Tata steel, 2012). Steel mainly consists of iron and by adding small quantities of other elements, the properties of steel undergo noticeable changes. Therefore the chemical composition of each steel specification is carefully balanced and tested during the production stage (Tata steel, 2012).

The manufacturing process of both the sheets and the members itself contribute towards the properties of CFS members as the members originate from the steel sheets. During the manufacturing of steel sheets, the milling process causes the grain structure of the steel to elongate. Further cold working of the steel also disturbs the grain of the steel. Depending on whether this distorted grain is repaired, through a process of recrystallization, the material properties undergo change. As part of the manufacturing process, the sheets need to be uncoiled and flattened to produce the steel sheets used to form profiled sections. In this process of



coiling, uncoiling and flattening, the physical properties of the steel undergo change (Hancock, 1996). During the cold forming of the profiled members, the steel properties are also influenced. From a structural point of view, the most important properties of thin-walled CFS are as follows (Wei-Wen, 2000):

- Yield strength
- Ultimate tensile strength
- Stress-strain characteristics
- Ductility

Referring to Figure 2.16 (typical stress-strain curve), the yield strength is described as the point at which steel starts to deform plastically. Before reaching its yield point, steel deforms elastically and is able to return to its original position once the load is removed. When the load is not removed and the stress passes the yield point, some permanent deformation will occur. For most steels, the elastic range is more or less the same and follows a linear curve, but once moving into the plastic range, it becomes non-linear and more dependent on the ductility and other properties. When it comes to the behaviour of CFS members under loading, it is known to be anisotropic. What this means, is that for different crystallographic orientations, the strength of the steel is different. Regarding the rolling direction, steel is known to have the highest yield strength in the transverse direction (Hancock, 1996).

When steel is cold-formed, the yield strength in the corners and flanges undergo an increase. This increase due to strain hardening depends on the type of steel that is cold-formed. Generally, the bigger the ratio of the ultimate tensile strength to the yield stress, the larger the effect of strain hardening is. In the case of 550 MPa grade steel, the steel yields gradually with minimum strain hardening. For the more ductile grade 300 MPa steel, more strain hardening occurs after a yield elongation plateau (Hancock, 1996). On the contrary, the increase in the ultimate strength is related to strain aging which is accompanied by a decrease of the ductility. Ageing of steel occurs when steel is held at ambient temperatures for several weeks or when held at higher temperatures for shorter time periods (Hancock, 2007).

As mentioned earlier, the grain structure of the steel gets distorted during the manufacturing process. This distortion causes an increase in the material strength and at the same time, a decrease in the material ductility. Through a process of recrystallization, the distorted grain can be restored. This however is only done for grade 300 MPa steel sheets and not for 550 MPa grade steel. By doing this, some ductility is restored. This is the reason for the 300 MPa grade steel having a higher ductility than the 500 MPa grade steel (Hancock, 1996). The difference in



ductility is visible when plotting a stress-strain graph for both these steel grades (Figure 2.16). The more ductile 300 MPa grade steel has an evidently higher tensile strength than yield strength where the two are more or less the same for the 550 MPa grade steel. This low level of ductility of 550 MPa grade steel is taken into account during the design stage (see Section 2.7).

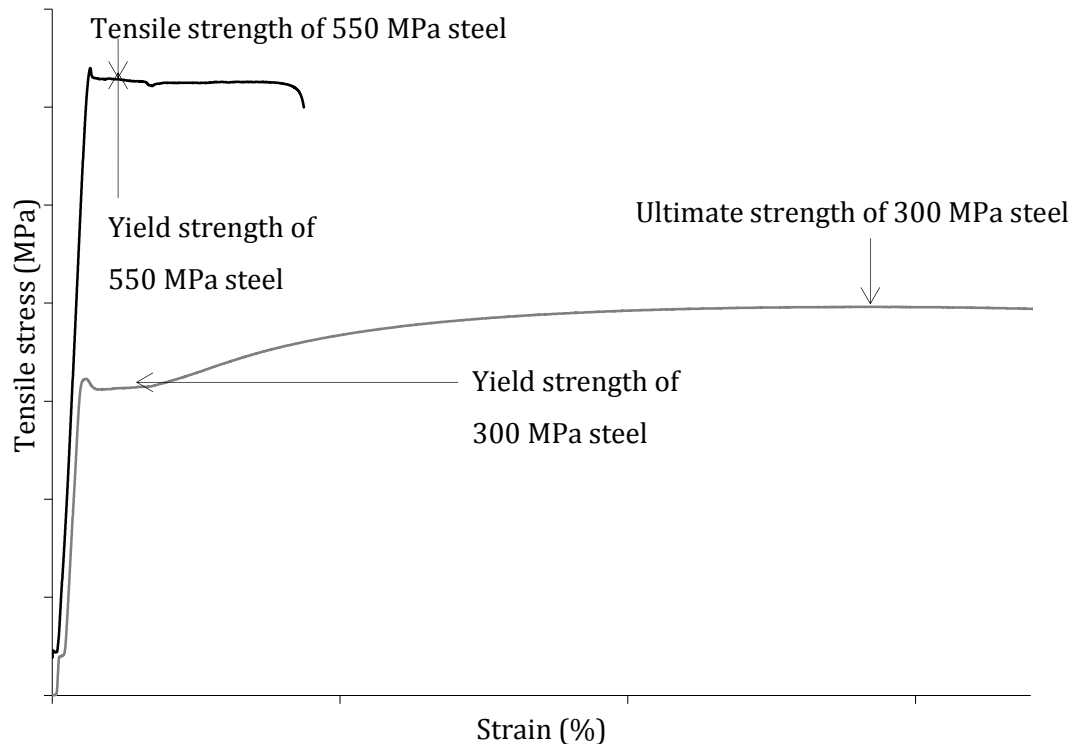


Figure 2.16 Stress-strain curve

Ductility can be defined as a material's ability to undergo sizeable plastic deformation without undergoing fracture. Not only is it required in the forming process, but it is also needed for plastic redistribution of stresses in connections and in the connected members (Wei-Wen, 2000). As discussed above, the ductility of CFS is reduced during the manufacturing process, thus the failure of CFS becomes more of a brittle failure depending on the amount of cold work performed on the material (Hancock, 2007).

CFS would typically have a designation of the form G550-Z175 where the first symbol (G) indicates that the mechanical properties have been achieved by in-line heat treatment prior to hot dipping. The three digit number that follows this symbol is an indication of the minimum yield stress in MPa taken in the rolling direction. The second part of the designation denotes a Zinc coating (Z) with the coating mass, 175, measured in grams per square metre of sheet. In order to protect the steel against corrosion, steel is often coated with some sort of coating mass. According to the ASTM A879 / A879M – 12 design standard, this coating process does not affect the mechanical properties of the base metal (Hancock, 2007).



When it comes to connecting two or more CFS members, strength, stiffness and ductility has the biggest influence. The connections together with the corresponding zone of interaction between the connected members are known as a joint. Structural joints can be classified into several categories referring to its strength and stiffness (Lee, 2014). According to Eurocode 1993 Part 1.8 Clause 5.2.3, the strength of joints can be divided into nominally pinned, partial strength and full strength classifications. According to Clause 5.2.2 of the same document, the stiffness of joints can be divided into nominally pinned, semi rigid and rigid classifications. In general, bolted and screwed connections are either rigid or pinned (Lee, 2014).

The strength of connections depends on the type of fastener and the connected member's material properties. Connections must possess the necessary strength to be able to resist the applied load and transfer the internal forces in between connected elements. Any structure is highly dependent on its connections between members, not only because it is the connections that keep it together, but because the stiffness of connections determine the overall stiffness of the structure.

It is important that the real behaviour of connections is taken into account in thin walled members; otherwise it can lead to serious unsafe and incorrect design. To prevent brittle failure, ductile behaviour of connections is necessary in order to allow local redistribution of forces without detrimental effects. To ensure this sufficient deformation capacity, the correct fastener, detailing and design is necessary (Dubina, 2012).

2.7 Connection design

2.7.1 General remarks

In structural design, the main purpose is to achieve acceptable probabilities that a structure or part of structure will not become unfit for its intended use. In other words, structures are designed not to reach certain states at which it is no longer feasible. These states are better known as limit states.

2.7.1.1 Limit state design

Limit state design provides the basic framework within which the performance and probability of failure of a structure can be assessed against various limiting conditions. In order to determine such a limit state, the variability of the materials, loads, construction practices and design approximations must be taken into account. One of the central concepts of limit state



design is that the design should be based on statistical methods with a small probability of the structure reaching limit state (Dubina, 2012).

This concept also implies the recognition of the fact that:

- The nominal values assumed for loads, material strengths, etc. in design are different in reality.
- Imperfections are introduced during fabrication and erection.

The effect that these factors have on the strength of the structure can only be realistically assessed in statistical terms. Uncertainties in loads, material strengths, etc. are taken into account through the use of partial safety factors (Dubina et.al, 2012).

Considering steel structures, there are four types of limit states:

- Ultimate Limit State (ULS)
- Serviceability Limit State (SLS)
- Fatigue Limit State (FLS)
- Accidental limit state (ALS)

ULS typically represents the collapse of a structure due to structural stiffness and stability where SLS typically represents a state where the structure is declared unfit for normal use but do not indicate the collapse of a structure. To be safe, all of the limit states must be taken into consideration and the best way to do this is to design for ultimate loading and then check that the remaining limit states' requirements are met. The general condition of structural reliability with respect to both ULS and SLS is that a structure must have the necessary capacity to resist all loads expected to act on it during its service life. Thus condition can be expressed as Equation 2.1.

$$Demand < Capacity \quad (2.1)$$

For connections in structures, the resistance is determined on the basis of the resistance of the individual mechanical fasteners, welds and the other components of the connection. Elastic analysis is recommended for the design of CFS connections. In connections subjected to shear where stiffeners are present with different stiffness, the fastener with the highest stiffness must be designed to carry the load of the connection (Dubina et.al, 2012).

When it comes to CFS structures there are five general problems that are connected to the ULS design criteria. Out of these five listed below it is the last point that is addressed in this thesis (Dubina et.al, 2012).



- Local instability and strength of sections.
- Interactive instability and influence of specific imperfection.
- Reduced capacity with reference to ductility, plastic design and seismic resistance.
- Fire resistance.
- Connecting technology and related design procedures.

2.7.1.2 General design considerations

Section 5 in the SANS 10162-2:2011 provides the design standards for mechanical and welded connections in CFS structures. In order to ensure an effective and successful connection, the correct connection must be chosen. By using the structural requirements, the appropriate connection can be determined (Toma et.al, 1993). The following structural requirements are vital in this decision making:

- Strength
- Stiffness
- Deformation capacity

Once the type of connection is identified, the number of fasteners or the fastening area can be determined by using the non-structural requirements of the connection. Typical non-structural requirements are water tightness, durability, aesthetics and economic aspects. Economic aspects, such as

- Number of fastenings
- Costs of fastenings
- Skill required
- Direct and indirect labour costs
- Maintenance cost

When it comes to the design of CFS structures, there are a few general design assumptions that have to be considered (Dubina et.al, 2012). These assumptions are now listed:

1. Connections must be designed based on realistic assumptions of the distribution of the loads through the elements of the connection and equilibrium must be maintained.
2. When the redistribution of internal forces within the connection is taken into account, allowance for the ductility of a connection may be made.
3. Residual stresses due to the fastening of the fasteners do not need to be considered.
4. Ease of erection and fabrication shall be accounted for during the detailing of connections.



5. Sufficient clearance necessary for the fastening of fasteners shall be taken into account during the design stage of structures.
6. The determination of any structural property of fastener or connection shall be based on design assisted by testing.

In general, it is ideal to design a connection so that the centroid axes of the connected members meet at a point, thus with no eccentricities present. However, in cases where this is not the case, the moments caused by these eccentricities must be designed for. For connections subjected to vibration forces or connections in shear that are subjected to reversal of stresses, either preloaded bolts, fit bolts or welding should be used (Dubina, 2012).

2.7.2 Design standards

All tests that were done throughout this project were done according to the necessary specifications given in the South African national design standards. As background to the experimental program (Chapter 5), the applicable specifications given in the SANS 10162-2 and SANS 517 are discussed in this section. The discussion only includes screwed connections subjected to shear as this is the type of connections that were tested.

2.7.2.1 Steel specifications

According to SANS 517: CFS components must comply with the requirements of the national standards for continuous hot-dip zinc-coated carbon steel of structural quality (SANS 4998). They must also have a coating at least equivalent in corrosion resistance and robustness to 200 g/m² galvanising (Z200) or a 150 g/m² aluminium-zinc coating (AZ150).

The resistance of connections to loads can be determined through either calculations, testing or a combination of both where the design capacities shall be determined in accordance with SANS 10162-2. The thicknesses used for calculation purposes are the Base Metal Thickness (BMT) which according to the SANS 517 is calculated as follows (SABS, 2011).

$$\text{Base metal thickness} = \text{Coated metal thickness} - 0,04 \text{ mm} \quad (2.2)$$

Regarding the manufacturing of the components, a $\pm 5\%$ tolerance is allowed for relevant actual sectional properties. A further 2 mm difference is allowed for the lengths of components and components that are specified to be straight shall not deviate about any axis from a straight line drawn between the end points by more than the lesser of $L/1000$ or 6,0 mm (SABS, 2011).

According to SANS 10162-2: Structural members or steel used in manufacturing is applicable if it appropriately complies with the SANS 4998. The SANS 4998 is the design standard for



continuous hot-dip zinc-coated steel sheets. According to the SANS 10162-2, 550 MPa grade steel may only be used if the steel thickness is equal to or greater than 0.9 mm unless the tensile strength used in design for connections are taken as the lesser of 90% of the specified value or 495 MPa. When the thickness is less than 0.6 mm, the tensile strength used in design must be taken as the lesser of 410 MPa or 75% of the specified value (SABS, 2011). These adjustments are done to take the low ductility of 550 MPa grade steel into account.

Tests done for the determination of the steel yield stress and tensile strength of the full section for axially loaded tension members must be determined as specified in Section 8 of the SANS 10162-2 and the SANS 6892. The minimum yield stress (f_y) and the tensile strength (f_u) used in the design are not allowed to exceed the values given in Table 2.1 (SABS, 2011).

Table 2.1 Minimum strengths of steels (SANS 10162-2)

Applicable Standard	Steel Grade	Yield stress (f_y) MPa	Tensile strength (f_u) MPa
SANS 4998	G250	250	320
	G300	300	340
	G350	350	420
	G450	450	480
	G500	500	520
	G550	550	550

Normally a certified mill test report or certificate issued by the mill shall provide evidence of compliance to the appropriate design standards. The uncoated thickness at any point of the member must be at least 95% of the value used in the design except for the thicknesses at corners due to cold forming effects (SABS, 2011).

2.7.2.2 Fastener specifications

SANS 517: Fasteners, connectors and fixing methods used for CFS structures must comply with all parts of the SANS 1700 (SABS, 2011).

SANS 10162-2: Screws are applicable if it complies with AS 3566.1 and AS 3566.2 (SABS, 2011). Screws used for CFS connections subjected to shear have to have shear strength of 1.25 times the nominal bearing capacity. Only self-tapping screws with nominal diameter (d_f) are applicable to the SANS 10162-2 where $3 \text{ mm} \leq d_f \leq 7 \text{ mm}$. The screws may either be thread-cutting or thread-forming, with or without a self-drilling point.

2.7.2.3 Connection specifications

According to the **SANS 517**, connections shall be designed to satisfy the following:

- Resist loads in the connection as a result of design load effects in connected members.
- Deformation shall be within acceptable limits.



- Appropriate allowance shall be made for any eccentricity and local effects at the connection.
- The uplift forces due to wind shall be assessed.
- The strength and serviceability of a connection shall be assessed by computation using SANS 10162-2.

SANS 10162-2: Any suitable fastening system, such as welding, bolting, screwing, riveting, clinching, nailing, structural adhesive or other mechanical means may be used to connect members. The minimum distance between centres of screws shall provide clearance for screw washers but may not be less than three times the nominal bolt diameter (d_f). The distance from the centre of a screw to the edge of any part may not be less than 1.5 times the nominal diameter.

2.7.3 Design capacities according to SANS 10162-2

The design connection capacity is the available capacity of a connection to be used for design purposes. The design capacity is obtained by multiplying the nominal capacity by a factor that takes uncertainties and variabilities of the nominal capacity into account (Wei-Wen, 2000). Such a factor is better known as a capacity reduction factor ϕ . This can be mathematically expressed by Equation 2.3.

$$\text{Design capacity} = \phi \times \text{Nominal capacity} \quad (2.3)$$

2.7.3.1 Nominal capacities

Nominal capacities are determined by calculations using specified material properties and dimensions derived from tests and/or accepted structural mechanic principles. The nominal capacities of screwed connections subjected to shear are given here and are categorised according to the failure modes expected to occur when the corresponding limit state is exceeded (Wei-Wen, 2000).

End tear out

$$V_f = t e f_u \quad (2.4)$$

Net section tension failure

For a single screw, or a single row of screws perpendicular to the force:

$$N_f = \left(\frac{2.5d_f}{s_f} \right) A_n f_u \leq A_n f_u \quad (2.5)$$



For multiple screws in the line parallel to the force:

$$N_f = A_n f_u \quad (2.6)$$

Tilting and bearing

Where the screw is in a single shear connection and where the two connected sheets are in contact at the point of fastening the nominal capacity is taken as the smallest of the following:

(a) For $t_2/t_1 \leq 1.0$

$$V_b = 4.2 \sqrt{t_2^3 d_f f_{u2}} \quad (2.7)$$

$$V_b = C d_f t_1 f_{u1} \quad (2.8)$$

$$V_b = C d_f t_2 f_{u2} \quad (2.9)$$

(b) For $t_2/t_1 \geq 2.5$

$$V_b = C d_f t_1 f_{u1} \quad (2.10)$$

$$V_b = C d_f t_2 f_{u2} \quad (2.11)$$

(c) For $1.0 \leq t_2/t_1 \leq 2.5$ the nominal capacity is determined by linear interpolation between the minimal values obtained at (a) and (b). The thickness of the connected member in contact with the screw head is denoted as t_1 where t_2 is the thickness of the member not in contact with the screw head.

Screws in shear

According to the SANS 10162-2 the nominal capacity of screws can be determined through testing where the capacity may not be less than $1.25V_b$, where V_b is the calculated tilting and bearing nominal capacity. The reason for the multiplication with 1.25 is to ascertain that the connection fail in either section failure or tilting/bearing and not due to failure of the screw. Screw failure needs to be prevented because it is brittle and a more ductile failure is preferred (Hancock, 2007).

2.7.3.2 Capacity reduction factor

Design standards that incorporate a limit states philosophy as a basis for design, e.g. AS/NZS 4600 (SANS 10162-2) are dependent on capacity reduction factors to take unavoidable deviations in the nominal capacity into account (Wei-Wen, 2000). These deviations are usually the result of uncertainties and variabilities in material properties, cross section geometry and design methods. In some limit state design methods, a capacity reduction factor also takes into account the type of failure anticipated for the member. In the AS/NZS 4600 (SANS 10162-2) this



failure mode uncertainty is not included in the capacity reduction factor, but has been included in the formulas that determine the theoretical member strength (Hancock, 2007). The capacity reduction factor given in the SANS 10162-2 for screwed connections expected to fail due to tilting and hole bearing is $\phi = 0.5$.

2.7.4 Background of design equation for tilt and bearing

The first equations regarding CFS screwed connections were published as part of the 1987 edition of the European recommendations for the design of light gauge steel members. This code suggested that the design strength of a fastening system can be determined through statistical evaluation of test results or by appropriate formulae. The first of such formulas was given by this ECCS document in 1987 and then modified by Teoman Pekoz in 1990. Regarding the tilting and bearing failure mode formula (Equation 2.7), the initial version was given as

$$V_b = \alpha f_y t d_f, \quad \alpha = 3.2 \sqrt{\frac{t}{d_f}} \quad (2.12)$$

This design formula was modified by Pekoz as he compared the predicted capacities to actual tested capacities. By substituting the yield strength with the ultimate shear strength in Equation 2.12, he found that the latter resulted in a much better correlation when compared to the tested values. The ratios of the tested values over the predicted values when using Equation 2.12 resulted in values significantly higher than one. When a factor of 1.3 is multiplied with the above expression of V_b , the ratios were much closer to one. The reason why a ratio is better when it is closer to one is because a value equal to one is an indication of a formula capable of precisely predicting the actual capacity. By substituting f_y with f_u and by multiplying the formula with a factor of 1.3, Equation 2.12 became Equation 2.7 which is currently used.

2.8 Previous relevant studies

2.8.1 Introduction

Whenever a new material enters the engineering world, the chance of it being immediately implemented is slim. The nature of human beings is to only use and implement something that has been proven to work and that is well known. CFS is still relatively new in South Africa and is therefore not yet seen as a primary building material and is still very much unknown to the general public. Due to its newness and unfamiliarity not a lot of research has been done on the use of CFS in South Africa. However, in countries more familiar with CFS, numerous research projects have been done on the structural use of CFS. Some relevant studies are discussed in this



section with the focus on the objectives, results and recommendations of these studies.

The motivation behind these discussions was to find evidence that encourages the current investigation and to have a broader understanding of how CFS behaves during certain tests. A large number of studies were investigated, but only typical studies are summarised here. The focus points of the respective studies were used as the discussion headings. Under each of these headings, the corresponding study is summarised with its relevance to the current research made clear under Section 2.8.6 where the important points of the respective studies are highlighted.

2.8.2 Inaccurate prediction model

A paper by Teh et al showed that the design formulation for bolted connections against net section failure in the SANS 10162-2 is mathematically incorrect and that it delivers unconservative capacity predictions. This study was based on the comparison between executed capacity tests and predictions made using the design formulation under consideration. More details are provided below.

The net section capacity of a bolted connection with a single bolt and no washers can be calculated using Equation 2.13. This is the equation given by the AS/NSZ 4600:2500 and SANS 10162-2. The term enclosed in brackets is known as the shear lag factor. The capability of this equation to predict the true net section capacity was determined through some engineering mathematics and numerous experiments (Teh, 2012).

$$N_f = \left(\frac{2.5d_f}{s_f} \right) A_n f_u \leq A_n f_u \quad (2.13)$$

- s_f is the section width.
- d_f is nominal fastener diameter.
- A_n is the nominal cross sectional area of the connected members $((s_f - d_f)t)$.
- f_u is the ultimate tensile strength of the steel.

By studying Equation 2.13 it is clear that as soon as the shear lag term exceeds unity, the shear lag factor gets neglected. Another observation made is that when the section width is reduced by an increase in the nominal bolt diameter, the net section capacity increases. This is in contrast with what is rationally expected and with the results found by the experiments done in this study. This flaw is emphasized even further by differentiating the capacity with respect to the nominal diameter. This is given by Equation 2.14 which shows that the net section capacity will only reduce with an increasing diameter when the section width is less than two times the diameter. In practice, the width is mostly larger than two times the diameter (Teh, 2012).



$$\frac{\partial N}{\partial d_f} = 2.5f_u t \left(1 - \frac{2d_f}{s_f}\right) \quad (2.14)$$

Tests were done on single bolted connections, connections with multiple bolts in a line parallel to the force and connections with a row of bolts perpendicular to the line of the force. Some specimens were loaded concentrically and others eccentrically. Grade G450 steel was used with two different nominal thicknesses of 1.5 mm and 3 mm. According to the test results, the equations in the codes more often than not over predicted the net section capacity. When Equation 2.13 was used for the capacity predictions, an average model factor of 0.95 was obtained with standard deviation of 0.025 (Teh, 2012). The model factor is defined as the ratio of the tested capacity over the predicted capacity and a model factor less than one is an indication of an unconservative prediction model. An alternative model for determining the net section capacity of a connection was proposed and is given here as Equation 2.15. This equation clearly shows that due to the fact that the specimen width will always be larger than the nominal bolt diameter, the shear lag factor will never exceed unity and the net section capacity will reduce for an increase in fastener diameter (Teh, 2012).

$$N_f = \left(0.9 + \frac{0.1d_f}{s_f}\right) A_n f_u \quad (2.15)$$

In conclusion, this paper found that Equation 2.13 sometimes gives a shear lag factor that exceeds unity which is not possible. Following the proposed method of determining the net section capacity of bolted CFS connections, the results were used to re-calculate the capacity reduction factor. As mentioned, a model factor of less than one was obtained when using the original design equation for the capacity predictions. When Equation 2.15 was used to predict the connection capacities, more reliable results were obtained. An average model factor of 1.02 with a standard deviation of 0.026 showed that the new equation is more conservative than the original capacity equation. A graphical comparison of the two capacity equations is shown in Figure 2.17 where the blue points (circles) represent the model factors obtained from using the adjusted prediction model (Teh, 2012).

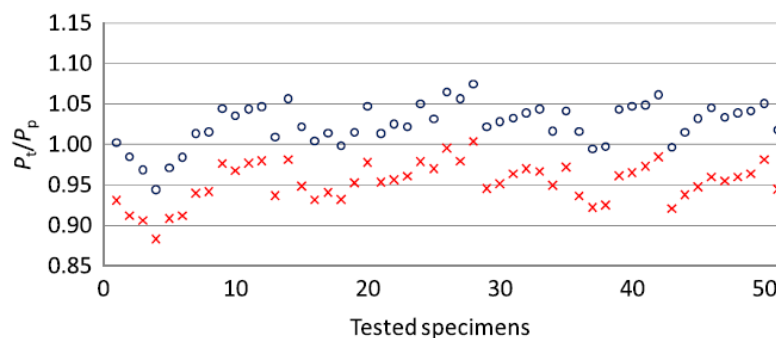


Figure 2.17 Eq 2.13 (red crosses) vs. Eq 2.15 (blue circles) (Teh, 2012)



2.8.3 General behaviour of screwed connections

Acknowledging the fact that screwed connections have received more and more attention in recent years, a study by Yuanqi focussed on the typical behaviour of screwed connections. The aim was to determine the effect of various connection properties on the shear strength and modes of failure (Yuanqi, 2010). The connection properties investigated are listed below.

- End distance
- Screw spacing
- Number of screws

The possible failure modes of typical screw connections are

- Screw tilting
- Hole bearing
- Edge tear out
- Failure of screw in shear
- Fracture in net section of connected elements

The experiment involved 75 single lap screwed connection specimens, all with a thickness of 1.0 mm. The width and the screw arrangement varied among specimens. The general result of these tests was that the strength of screw connections is affected by all of the investigated connection properties (Yuanqi, 2010).

2.8.3.1 Connection properties

End distance

A total of 9 connection specimens were tested to determine the effect of the end distance on the strength of the connections. The end distance was altered from $2d$ ($2 \times$ nominal diameter of fastener) to $4d$. The shear strength was the lowest for an end distance of $2d$ and increased slightly for larger end distances. When the end distance was equal to or more than $3d$, the end distance seemed to have little influence on the connection strength (Yuanqi, 2010).

Screw spacing

A total of 5 screws were used with the spacing between the screws changed to see whether it affects the connection strength. The five different screw spacings used included $3d$, $4d$, $5d$, $10d$, $15d$ and $20d$. The connection strength increased up to a spacing of $5d$ after which no real difference was found. This is shown in Figure 2.18.

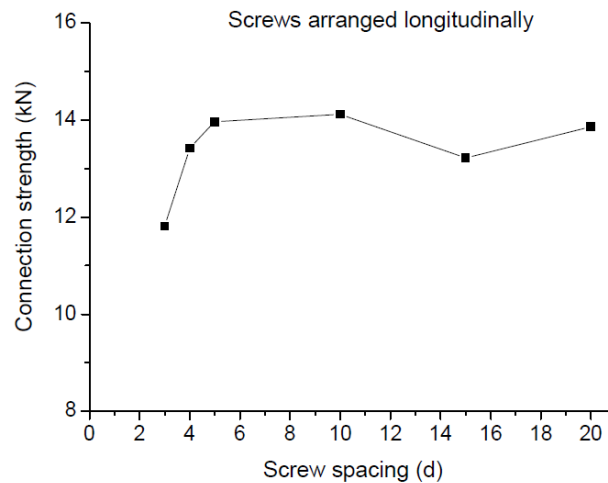


Figure 2.18 Effect of screw spacing on shear strength (Yuanqi, 2010)

Number of screws

A clear increase in the shear strength was observed as the number of screws was increased. No matter how the screws were arranged, the strength per screw diminished as the number of screws increased. This means that the strength of a connection with four screws is less than four times the strength of a similar connection containing one screw. This is known as the “group effect” (Yuanqi, 2010).

2.8.3.2 Failure modes

Out of the 75 single screw tested specimen, 64 failed due to the combination of screw tilting and plate bearing. The accuracy of the design formula (AS/NSZ 4600:2500) used to predict the capacity of screwed connections expected to fail in tilting and bearing were also tested. It was found that the predicted shear capacities for single screw connections agreed well with the corresponding tested capacities. An average model factor (ratio of tested capacity over predicted capacity) of around one was obtained for connections containing one screw. As the number of screws increased the model factor became unconservative with model factors reaching averages as low as 0.858 (Yuanqi, 2010).

2.8.4 Misidentification of failure modes

Misidentification of failure modes can lead to serious errors in the accuracy and applicability of design equations. Rogers and Hancock did a study that focussed on the physical failure mode identification of tested specimens (Rogers, 2000). One of the main misconceptions is the occurrence of small tears in most of the sheet failure modes. Sometimes when cracks are observed perpendicular to the force, it is assumed that the net section have failed where in fact the cracks may have been caused due to the localised bearing stress as the steel piles up in front



of the bolt. According to this study the following characteristics can be used for accurately identifying net section failure and bearing failure (Rogers, 2000).

Net section failure:

- Necking across the width of the specimen takes place.
- Tears originate from the centre of the originally drilled hole.
- The fastener hole furthest away from the end undergoes failure.
- Significant out of plan curling of the specimen does not occur.
- Significant out of plane sheet deformation in the vicinity of the plane does not occur.

Bearing failure:

- Significant out of plane curling of specimen end.
- Significant out of plane deformation in the vicinity of the fastener.
- No necking across the thickness of the test specimen.
- Tears originate near the material pile in front of fastener or washer and not from the centre of the originally drilled hole.

2.8.5 Experimental set-up example

Most of the studies done on the behaviour of screw connections subjected to shear more or less made use of the same experimental set-up. Rogers and Hancock (1999) did numerous tests on screwed connections of thin G550 and G300 sheet steels. Before getting to the discussion of this experimental procedure, it is worth mentioning that all of the 150 tested connections failed in a combination of screw tilting and hole bearing. It was also concluded that this type of failure was due to the thinness of the members (0.42 mm, 0.6 mm, 0.8 mm and 1.0 mm) (Hancock, 1999).

The experimental set-up usually involved two steel strips of similar size that were connected with a screw or set of screws. The steel strip dimensions and screw diameters differed among the various studies. For the tests done by Rogers and Hancock, the steel strips had the dimensions shown in Figure 2.19. The testing apparatus (Figure 2.20) was designed to eliminate slippage at the grips and to transfer the load evenly through the specimen cross section. The ultimate load and the displacement at the ultimate load were obtained. The ultimate load was taken as the highest force reached before a connection displacement of 6.35 mm. Displacement transducers, situated around mid-height of the specimen in Figure 2.20, were used to measure the displacement of the connected members relative to each other. Although the centreline of the actual testing machine and the centreline of the specimen does not line up in Figure 2.20, the reference Hancock et.al clearly states that this eccentricity was equal to zero when screwed



connections were tested. The basic material properties for the sheet steels were obtained through tensile tests according to ASTM A370 which proposes a somewhat similar tensile test as the SANS 6892.

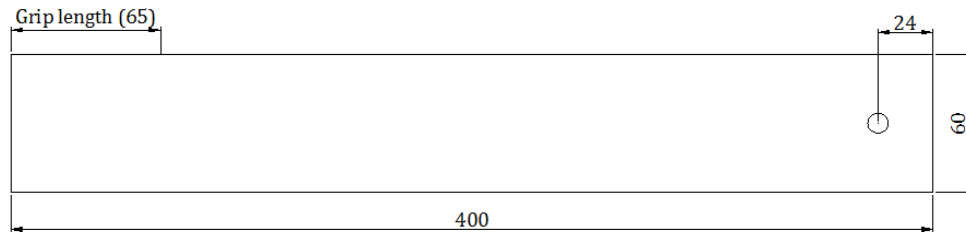


Figure 2.19 Test specimen used by Rogers and Hancock

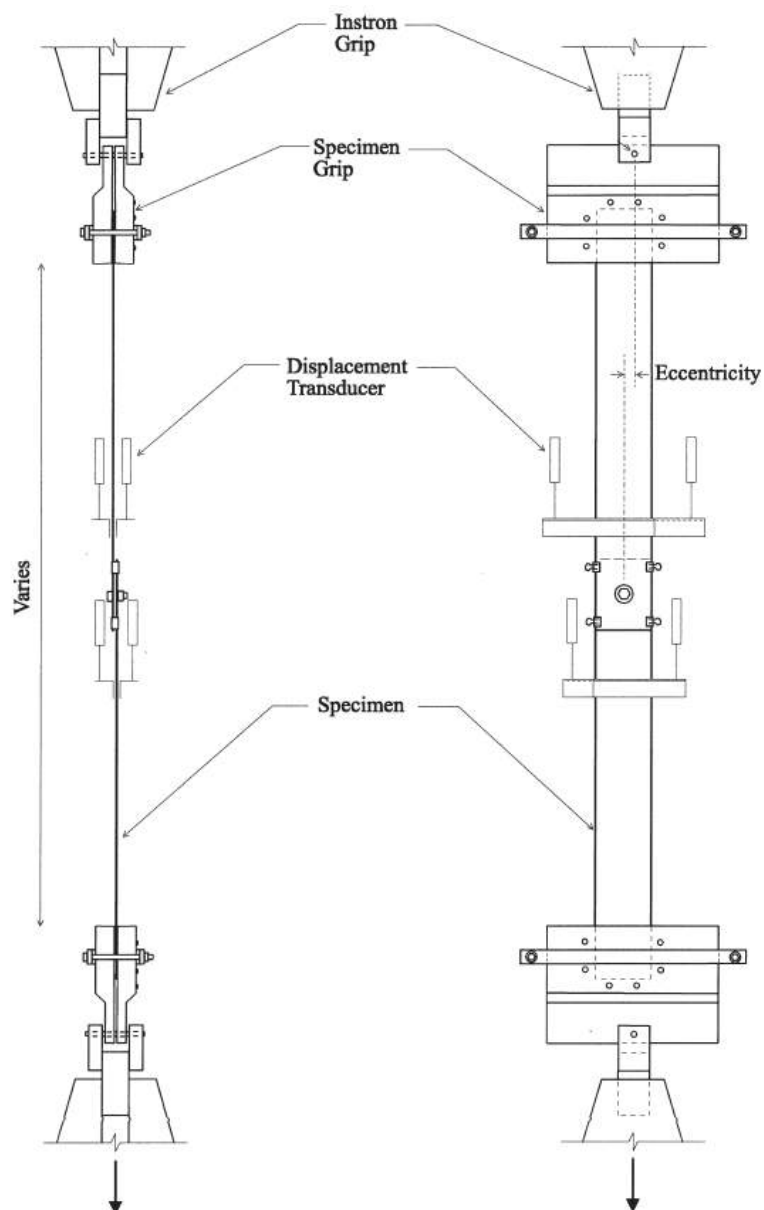


Figure 2.20 Typical test set-up (Hancock, 1999)



2.8.6 Concluding remarks

The first study discussed is typical of studies that have evidence of faulty formulations in the SANS 10162-2 design standard. Although the design formulation under discussion predicts the net section failure of bolted connections, the same design standard is used for screwed connections. Any incorrect formulation of the SANS 10162-2 contributes to declaring the standard as untrustworthy. In conclusion, this study by Teh (2012) found a faulty formulation in the SANS 10162-2 and thus leaves room for doubting the correctness of other similar formulations.

The amount of screws in a CFS screwed connection can vary depending on where and what it is used for. The second study gave some information of connection properties and their effect on the shear strength. It also showed what the most common failure mode is for the different properties. It was evident that for a connection that only had one screw, the most common failure mode was tilting and bearing. The relevance of this study was the failure modes of single screw connections as tilting and bearing is the focus of the current research. Failure modes of screwed connections are often misidentified. This was the motivation behind the third study discussed. This discussion concluded with a set of recommended characteristics that can be used for distinguishing between failure modes.

Relevant between the studies investigated and the current study is the experimental set-up. Some guidelines of the experiments done by the various research parties were obtained and implemented. Some problems and obstacles experienced could be prevented by using the recommendations given in the previous relevant studies.

Chapter 3

Structural Reliability

3.1 Introduction

Up until two decades ago structural design was done in a deterministic manner. Elements were designed to have a strength that exceeded the load with a certain margin where the ratio of the strength and the load was considered as a measure of reliability (Sorensen, 2004). These methods of assessing various uncertainties were insufficient and led to the need for a different approach. This came in the development of the structural reliability theory. The reliability theory is a probabilistic design approach that assesses uncertainties associated with structures in a rational manner. This chapter gives some background on structural reliability and its design methods after which the process of determining the reliability status of a structure is discussed.

3.2 Structural design methods

In the field of civil engineering, there are three basic methods that are used for design purposes. These three methods, with various modifications, are being applied in various standards for structural design (Holicky, 2009). The most recent and most advanced design method is the partial factor method. The partial factor method is also known as the limit state design method and is based on the following condition:

$$E_d(F_d, f_d, \theta_d) < R_d(F_d, f_d, a_d, \theta_d) \quad (3.1)$$

where E_d and R_d is the action effect and resistance effect respectively. Both of these are assessed according to the design values of basic quantities describing the action F_d , material properties f_d , dimensions a_d and model uncertainties θ_d which are determined using the characteristic values, partial safety factors, combination factors and other measures of reliability. The design values take their uncertainties into account as well as the computational model uncertainties where the probability of the structure is dependent on a whole system of



partial factors (Holicky, 2009). The other two structural design methods are the method of permissible stresses and the method of global safety factor. The main insufficiency that makes these methods less popular than the partial factor method is that it is not possible to consider uncertainties of basic quantities and of the computational method separately (Holicky, 2009). The method of permissible stresses is based on

$$\sigma_{max} < \sigma_{per}, \text{ where } \sigma_{per} = \sigma_{crit}/k \quad (3.2)$$

Where k symbolises the uncertainties of both the local load effect σ_{max} and the resistance σ_{per} . As these uncertainties are taken into account it may be said that with an appropriate level of surety, the reliability of the structure is ensured. As mentioned, the insufficiency of this method is that there is only one value that takes the uncertainties into account and thus the probability of failure is controlled by k only. Based on the following condition, the method of global safety factor is somewhat similar to the permissible stresses method (Holicky, 2009).

$$s = X_{resist}/X_{act} > s_o \quad (3.3)$$

To ensure the reliability of a structure using this method the calculated safety factor s must be greater than its specified target value s_o . Again, the probability of this method is only dependant on one value - the safety factor (Holicky, 2009). If one of these three design methods discussed were to be used to determine the structural reliability of structures made of different materials the partial factor method seems to be the most adequate.

To account for uncertainties in a more sufficient manner and to improve the measures of structural reliability, methods based on probabilistic techniques have to be used. By using reliability theory as basis for structural design, the uncertainties associated with structures are assessed in a rational manner. This is a probabilistic design approach where the probability of failure is directly applied. The probabilistic design approach is discussed in Section 3.4.

3.3 Uncertainties

Construction works are known to be technical systems that suffer from significant uncertainties at all stages of execution and use. Some uncertainties can never be neglected and must therefore be accounted for during the design stage. Depending on the nature of the structure, environmental conditions and applied actions, some uncertainties may become critical. Based on how the uncertainties are handled, it can be separated into two categories: Aleatory and epistemic. Aleatory uncertainty is that uncertainty related to luck or chance that arises from randomness inherent in nature where epistemic uncertainty is the uncertainty related to



knowledge. The factors that cause these uncertainties can be grouped into the following sources (William, 2008; Holicky, 2009):

- Statistical limits: Insufficient amount of data.
- Model limits: Simplifications made to prediction model
- Randomness: Properties vary over some range and are thus seen as random variables.
- Human error

The way in which uncertainties in structural engineering are dealt with depends on the source of the uncertainty. If uncertainties can be identified they can be reduced by appropriate control measures. Uncertainties due to human error and insufficient data are examples of uncertainties over which humans have control and thus, through quality control, can be reduced (König et.al, 1985). The natural randomness of actions, material properties and geometric data is also uncertainties that can easily be identified (Holicky, 2009), and for which appropriate provision can be made in design codes.

3.3.1 Human error

Design errors and errors during construction are two types of human errors that cause uncertainties. Design errors often refer to simple miscalculations made during the design stage which is more likely to occur when complex design models are used. By simplifying the equations used in design, other uncertainties are induced. To minimise uncertainties, a good balance between the complexity and accuracy of the design formulae must be established. It is clear that these types of uncertainties cannot be taken into account through design codes, but must be dealt with through quality control. Quality control includes any technique that reduces the likelihood of error, such as design checks and inspection (William, 2008).

3.3.2 Natural randomness

The design of any structure or structural component is based on the nominal values of the contributing variables. Due to the random behaviour of variables, the actual values of the variables are often different from the nominal values. Although the actual values differ from situation to situation, nominal values are used for the parameters in design models (König, 1985). This randomness (uncertainty) can cause unacceptable variations in the actual strength of the structural part under consideration. Uncertainties caused by the natural randomness of material properties are often referred to as intrinsic physical or mechanical uncertainties.

This type of uncertainties are modelled and quantified by capturing the essential features of the uncertainties through random variable models. By adopting a suitable probability distribution



for each of the random variables, the uncertainties are being represented. Material properties and geometrical properties are examples of random variables usually present in typical resistance prediction models. Geometrical properties generally have a small coefficient of variation while material properties are known to have a greater variation. It is due to this inconsistency of properties, that an adequate probability distribution must be chosen to represent the variable and thus also its corresponding uncertainties (JCSS, 2000).

Even if the variation in material properties can be assessed, uncertainties will mostly remain present due to measurement errors and small data sets. Another uncertainty considered here is the uncertainty associated with the resistance model. Unlike the natural randomness of material properties, this type of uncertainty is not as well studied and understood. Denoted as model uncertainty, it is generally used as a random variable accounting for effects neglected in the mathematical simplifications and design models.

3.3.3 Model uncertainty

Throughout the design and testing of any structural component, various models are used which more or less represent the undesired state of the structure. These mathematical models are the basis of many decisions regarding structural reliability. The derivation of such models is not always known and thus a number of uncertainties are present. Model uncertainty is seen as having a special place as on the one hand they cannot be easily assessed and taken into account, and on the other hand they cannot in general be seen as negligible (König et.al, 1985). When real life decisions are to be made, uncertainties within the model itself must be considered and quantified. This quantification can only be done by comparisons with other more involved models that represent a closer representation of reality or by comparison with collected data from experimental work.

The incomplete understanding of the real behaviour of structures is one of the main causes of model uncertainties. Sometimes it is necessary to make various simplifications to the models used for design in order to prevent an increase in computational effort. However, this simplification mostly results in uncertainties due to the calibration of the simplifications made to the mathematical formulation of the model. Through practical experience, uncertain design models can sometimes be identified. If the structure's resistance is not as predicted by the appropriate design standards, this alteration can possibly be due to an uncertain design model. During the erection and use of a structure, measurements of the structure's strength, deformation, stresses etc. can be made and then be compared to values obtained by using the



mathematical design models. A deviation in the compared values can be a result of uncertainties (König et.al, 1985).

In structural design, a variety of prediction models are used to determine the resistance capacity and the possible load effects of a structure. There are uncertainties present in both these types of models. Model uncertainties depend significantly on the prediction model under consideration and the type of structural member the model is used for. Applicable to both resistance and action effect models, Figure 3.1 shows some of the general concepts of model uncertainties (Holicky et.al, 2013).

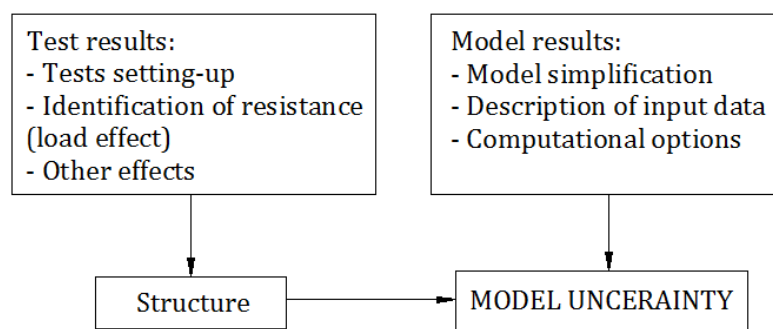


Figure 3.1 General concepts of model uncertainties (Holicky et.al, 2013)

To accurately predict structural resistance, all uncertainties need to be taken into account on a quantitative manner. For uncertainties associated with the resistance model, this quantification is done with the use of a unitless factor known as a model factor. A model factor is defined as a factor that represents all uncertainties associated with the model used for resistance predictions. Defined as a random variable, the model factor θ can be obtained by comparing actual test results with the results from prediction (resistance) models and thus have a multiplicative definition expressed as

$$R_{test} = \theta R_{model} \quad (3.4)$$

R_{test} is the resistance derived from actual tests where R_{model} is the unbiased resistance prediction. An unbiased prediction refers to a prediction made by neglecting all conservative bias, partial factors and safety elements incorporated for design purposes. It must be noted that although the tested capacities are taken as the true resistance, they too have some experimental error and uncertainty inhibiting them from being truly representative of the actual situation (Mensah, 2012).

Being the ratio of a tested value over a predicted value, an average model factor equal to unity is an indication of a model capable of producing unbiased predictions of the resistance capacity of a connection. Whenever these two values deviate from one another, uncertainties are present.



An example of a factor that influences this resistance model uncertainty is actual structural conditions not covered by the tests performed to obtain the actual resistance. Other examples of factors affecting the uncertainty related to resistance models are given below (Holicky et.al, 2013).

- Accuracy of test method
- Deviations of individual tests
- Vagueness in failure indicator
- Boundary conditions
- Size effect and other effects not covered by tests.

In general, the following aspects should be considered in the assessment of the uncertainty related to resistance models (Holicky et.al, 2013).

- The test conditions must be correctly defined.
- The test results should be properly evaluated.
- The definition of failure should be clear as the uncertainty of the resistance model depends on it.
- A specimen should always fail in an investigated failure mode.

The assessment of uncertainties have for many years been done on a deterministic manner. Some of the design methods that assessed the uncertainties in such a way were discussed in Section 3.2. When following this deterministic approach, the formulae used for design would have a large safety margin to account for all uncertainties. When this model delivered unsatisfactory results, the limit state function would be modified by adding a new parameter or by increasing the safety margin (König, 1985).

An alternative approach in designing structures and assessing uncertainties is to use the theory of reliability. Rather than working with deterministic values, the variables are interpreted as random and are represented using probability distributions. This probabilistic approach tests the sensitivity of the respective variables towards the overall structural reliability and allows for the individual assessment of a wider range of uncertainties. An example of such a model uncertainty assessment is given in Chapter 7 where the actual experimental data of this research and the resistance prediction models of SANS 10162-2 are used. A model factor is determined for typical cold-formed steel screw connections expected to fail in tilting and bearing.



3.4 Probabilistic design approach

In our daily lives, we use words such as risk, chance, likelihood and probability. This is an indication of being uncertain about the state of the item or issue under consideration. In everyday terms, the level of uncertainty are usually expressed as qualitative statements such as “the chance is good” or “the probability is low”, but when it comes to structural reliability it is rather quantified in terms of numbers or percentages (Faber, 2002). This quantification is done by generating the basic variables as statistical distributions.

The probabilistic design approach is based on probabilities and reliability targets rather than deterministic criteria. Unlike the three basic structural design methods discussed under Section 3.2, the probability of failure is applied directly when following a probabilistic approach. Probabilistic design methods are based on the condition that the failure probability of a structure P_f does not exceed a specified target level of probability P_t . The probability of failure can be assessed using the limit state of a structure defined by the performance function $g(\mathbf{X})$ where $\mathbf{X} = [X_1, X_2, \dots, X_n]$ represents basic quantities for actions, mechanical and geometric properties. This assessment of the probability of failure is better known as the reliability theory and is further discussed in the next section (Holicky, 2009).

3.4.1 Reliability theory

When it comes to defining structural reliability, the definition is often oversimplified and is used very vaguely. In the world of structural design, the concept of reliability is sometimes approached in an absolute manner that states that the structure either is or isn't reliable. When following this approach it is understood that when a structure is reliable, the failure of the structure will never occur. This oversimplified approach is incorrect as failure may occur even when a structure have been declared to be reliable. A better, more realistic definition of reliability is: *Structural reliability is the ability (probability) of a structure to fulfil required functions during a specified life time under specified conditions*. There are four key elements included in this definition of structural reliability (Holicky, 2009):

- Required functions- the definition of failure.
- Specified life time- the required service life.
- Reliability level-probability of failure.
- Specified conditions-limiting input uncertainties.

Civil engineering structures are built for various purposes, where in most cases, people will be in or on these structures relying on it being safe. The basic requirement for a structure to be



reliable is to have a resistance capacity that is bigger than any action effect on the structure. Structures are designed to meet a specific required reliability, called the reliability index (Holicky, 2009). Structural failure can be described as a structure not being able to resist the applied actions. This resistance of a structure depends on the material properties, structural geometry and the uncertainties associated with an applied resistance model. When considering two random variables, resistance R and load E , the probability of failure can be expressed as

$$p_f = P(E > R) \quad (3.5)$$

The reliability theory can be concluded as a probabilistic design approach that is primarily based on the probability of failure. In the process of determining this failure probability, the characteristics of the variables must be probabilistically quantified. This can be done by the collection and analysis of experimental data. This process of statistically analysing the different variables and then determining the probability of failure is often referred to as a reliability analysis. Another statistical parameter that is used during the reliability analysis is the reliability index, denoted as β . As part of a reliability analysis, the reliability status is determined by comparing the determined reliability measures to the target values (Sorensen, 2004). The reliability analysis procedures are discussed in more detail in the following section.

3.5 Reliability analysis

A reliability analysis is a probabilistic approach in determining the reliability of a structure or part of structure. After listing the key steps generally followed in a reliability analysis, it is thoroughly discussed using theory and various graphical illustrations. The purpose of these discussions is to give the necessary background of the general procedures followed as a clear understanding is vital when the performed reliability analyses are discussed in Chapter 7 (Sorensen, 2004). The key steps of a reliability analysis are:

1. Select target reliability measures (probability of failure and reliability index).
2. Identify significant failure modes (limit states).
3. Formulate limit state functions (LSF).
4. Identify the random variables and parameters in the limit state functions.
5. Specify the distribution types and statistical parameters used to represent variables.
6. Estimate the reliability measures of the limit state function (LSF).
7. Evaluate the reliability results by using the FORM method.



3.5.1 Target reliability measures

The requirements that must be met by any structure in order for it to be declared reliable can be expressed in terms of the accepted minimum reliability index or the accepted maximum probability of failure (JCSS, 2000).

In a rational analysis the target reliability is considered as a control parameter that assigns a particular investment to the material placed in the structure. As it is seen as an investment, it can be said that the more the material is put in (invested) in the right places, the less the expected loss would be. Such an optimisation is mainly possible when the primary concern is economic loss rather than the loss of life. When considering life loss as the primary component, the target reliability measures are chosen as such that the cost per life saved is at acceptable levels. In a practical approach, the required reliability of a structure is controlled by a set of assumptions about quality assurance and formal failure probability requirements. These requirements are defined by specified target reliability levels for the various classes of structures and structural members (JCSS, 2000).

The target reliability level is probabilistically specified in terms of the reliability index β_t which is related to the target probability of failure, $p_f = \Phi(-\beta_t)$, where Φ is the standardised normal distribution. In South Africa, Table 3.1 is used to determine the minimum target reliability measures for a 50 year reference (SABS, 2011).

Other than the classes specified in Table 3.1 there are two more reference classes that are specified in the SANS 10162-2. For structures with brittle and sudden modes of failure, the minimum level of reliability is $\beta_t = 4$ and for connections the minimum level is $\beta_t = 4.5$. The target reliability indexes were derived from previous reliability studies of structures made from different materials and are highly dependent on the assumed theoretical models used to describe the basic variables. Although these models have not been used systematically, the recommended reliability index values are generally seen as reasonable average values of the reliability level characterising the existing structures (Holicky, 2009).

The above discussed reliability indexes are seen as measures of the overall reliability of a structure or part of structure. By overall, it is meant that the load E and resistance R components are considered together, as a function. It is, however, routine to consider only load or resistance separately by recognising that each of these components contribute in part to the total reliability. Their respective contributions are determined based on sensitivity factors that can be calculated using Equations 3.6 and 3.7 (Holicky, 2009).



$$\alpha_E = \frac{-\sigma_E}{\sqrt{\sigma_E^2 + \sigma_R^2}} \quad (3.6)$$

$$\alpha_R = \frac{\sigma_R}{\sqrt{\sigma_E^2 + \sigma_R^2}} \quad (3.7)$$

It is clear that these equations are dependent on the relative standard deviations of the load and resistance variables. Thus, the contribution towards the overall reliability depends on the variability of the load/resistance variable. The Eurocodes have approximated and accepted values of $\alpha_R = 0.8$ and $\alpha_E = -0.7$ for the resistance and load sensitivity factors respectively. The minus sign is just to distinguish between the loading component and the resistance. These sensitivity factors are multiplied to the overall reliability to obtain the respective resistance and load reliabilities. This allows for comparing the computed resistance reliability to the target resistance reliability ($\beta_{t,R} = 0.8 * \beta_t$). Considering connections, the target reliability is $\beta_{t,R} = 0.8 * 4.5 = 3.6$ (Holicky, 2009).

Table 3.1 Reliability classification (SABS, 2011)

Reliability class	Function of facility, probability or consequence of failure	Examples	Minimum level of reliability (β)
RC1	Low for loss of human life, economic, social or small or negligible for environmental consequences	Agricultural buildings with infrequent human occupancy (for example, storage buildings, greenhouse)	2.5
RC2	Moderate for loss of human life, economic, social or considerable for environmental consequences	Residential and office buildings, public buildings where consequences of failure are moderate (for example, office buildings)	3.0
RC3	High for loss of human life, or extremely high for economic, social or environmental consequences.	Grandstands, public buildings where consequences of failure are high (for example, concert halls)	3.5
RC4	Post-disaster function or consequences beyond the boundaries of the facility	Hospitals, communication centres, fire and rescue centres	4.0

3.5.2 Limit states (functions)

The state of any structure can either be satisfactory or unsatisfactory where the conditions that separate these states are called the limit states. Thus, the limit states are those beyond which a structure no longer satisfies the design criteria. A limit state can be defined in a very precise



way by describing a structure to be fully satisfactory only up to a certain value of the load effect (Holicky, 2009). In general there are two types of limit states. Each of these limit states is associated with a given set of performance requirements to which it must comply in order to be satisfactory. These recognised limit states are ultimate limit state (ULS) and serviceability limit state (SLS).

3.5.2.1 Ultimate limit state

The ULS is associated with collapse and other similar forms of structural failure. When considering ULS, the first passage of limit state is usually equivalent to failure. In some cases, excessive deformations can also be treated as the ultimate limit state. The ultimate limit state is directly concerned with safety and in some cases the protection of goods (Holicky, 2009). Typical scenarios that are considered as ULS are listed below:

- a) Loss of equilibrium of a structure considered as a rigid body.
- b) Unstable structures.
- c) Failure of structure due to rupture, fatigue or excessive deformation.
- d) Transformation of a part of structure into a mechanism.
- e) Sudden change of the structural system to a new system.

3.5.2.2 Serviceability limit states

The SLS is associated with the functioning of the structure or its members. Where the ULS is concerned with the safety of the people, the SLS is more concerned with the comfort of people as well as the appearance of the structure. There are two types of SLS, the irreversible SLS and the reversible SLS. Irreversible SLS is when the limit state remains permanently exceeded even when the action that caused the infringement is removed. Reversible SLS then refers to cases where the limit state will not stay exceeded once the action that caused the infringement is removed (Holicky, 2009).

3.5.2.3 Limit state functions

For any structural component a boundary can be theoretically formulated that divides the states of failure and non-failure. This boundary is referred to as the limit state function (LSF) and is generally denoted as $g(x)$ where x represents the realisations of random variable X . Such a formulation usually has two or more components. Say, for discussion purposes, the LSF under consideration consist of two terms, the resistance capacity R and the load E . Then the limit state function can be expressed as, $g = R - E$. Both of these terms depend on random variables and can therefore be represented using probability density functions. When considering the LSF, these terms (E and R) combine and thus generate a joint probability distribution. By using this joint distribution the probability of failure of the LSF can be determined where the LSF is



represented by a line, dividing the area into a safe and an unsafe region. When a structural component is analysed it can either fall in the safe region where $g(x) > 0$ or it can fall in the failure region where $g(x) < 0$. Thus, the limit state function can also be referred to as the failure surface where $g(x) = 0$. These safe and failing domains are graphically shown in Figure 3.2.

The LSF is a relationship between numerous variables and any combination of variable values will result in a point somewhere on the area, either on the safe or on the unsafe side of the function line. When the two terms are equal, the result will, of course, be zero and fall on the line. This discussion is fairly simple when only considering two random variables and gets complex when the number of terms (variables) increases. The basic idea and meaning behind the LSF, however, stays the same.

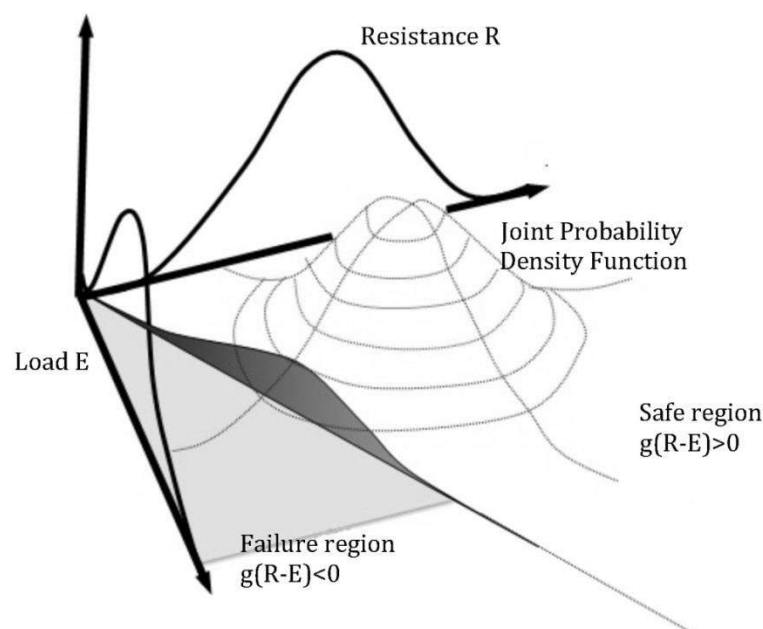


Figure 3.2 Limit state function (2R Software, 2010)

3.5.3 Random variables

A random variable is a function that assigns a unique numerical value to each outcome of a random experiment. As the experiment is repeated, this value of the random variable will vary. In structural reliability there are two types of random variables, continuous random variables and discrete random variables. Discrete random variables are variables that attain a distinct value where continuous random variables attain values within a given interval or domain (Montgomery, 2003; Easton, 2014). Continuous random variables are often applied to describe actions, material properties, and geometric data where discrete random variables are primarily used for counting the number of a certain event (Easton, 2014; Montgomery, 2003).



3.5.3.1 Distribution Probabilities

Discrete random variables are usually represented using probability distributions where probability density functions are used for continuous random variables (Montgomery, 2007). Say we denote a random variable as X and a realisation of this variable as x then the distribution and probability function of X can be expressed as Equations 3.8 and 3.9 respectively. The distribution function of X can be described as a function that gives the probability that random variable X is less than or equal to x . The probability density function is the derivative of the distribution function. It is defined as a function that can be integrated over a certain interval to obtain the probability that random variable X attains a value x in that interval. When using a normal distribution to represent random variable X , the probability of X attaining a value between a and b is equal to the shaded area in Figure 3.3.

$$\Phi(x) = P(X \leq x) \quad (3.8)$$

$$\varphi(x) = \frac{d\Phi(x)}{dx} \quad (3.9)$$

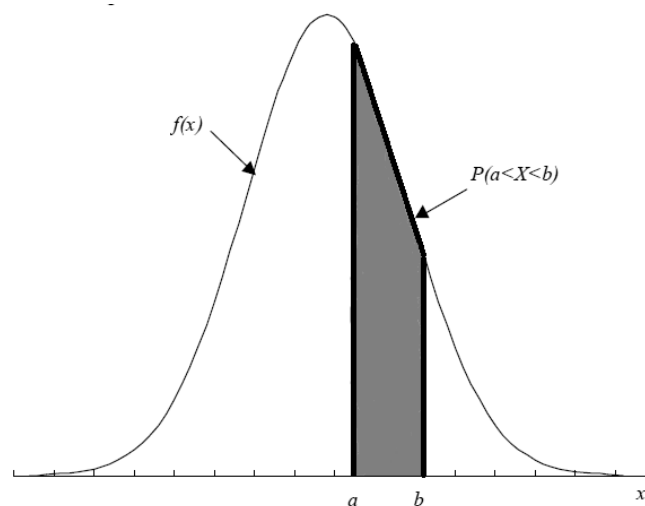


Figure 3.3 Example of probability computation (Lungu, 2007)

3.5.3.2 Statistical parameters

In addition to the distribution function and probability density function random variables can also be described by using various statistical parameters. The parameters most frequently used to describe random variables are moment parameters (Holicky, 2009). The moment parameters are:

- Mean (μ): Describes the location of a population
- Variance (σ^2): Measure the dispersion of a random variable
- Standard deviation (σ): Square root of variance
- Skewness (α): Measure of asymmetry



- Kurtosis (ε): Measure the steepness
- Coefficient of variation (w): $w = \frac{\sigma}{\mu}$

Besides the moment parameters listed, there is number of other parameters that are used to describe the distributions of random variables. Out of all these parameters, it is the mean, standard deviation and coefficient of variation that are mostly used to describe and characterise a variable distribution (Montgomery, 2003).

3.5.3.3 Standardised random variables

Up to now the determination of the probability has been associated with an integration process over a certain interval. This complicated process is not necessary, as an alternative exists in the form of a standardised random variable. By standardising a random variable, the process of integration is eliminated by using published tables that gives the probabilities for the calculated standardised random variables.

The most important distribution for a continuous random variable is the Normal distribution. A special case in which the distribution mean is zero and the variance equal to one is known as the standardised normal variable. In general, a normal distribution has a mean and a standard deviation that is used to standardise the variable. This transformation is done using Equation 3.10. The transformed standardised random variable Z has a mean $\mu_Z = 0$ and a standard deviation $\sigma_Z = 1$ (Lungu, 2007).

$$Z = \frac{X - \mu}{\sigma} \quad (3.10)$$

3.5.4 Distribution types

Depending on the properties of the random variables, a variety of probabilistic distributions are available for modelling the variables. The distribution types most frequently used in structural reliability are discussed in this section. As described earlier in this chapter, the probability density functions are used for the determination of probabilities and will therefore form the basis of these discussions.

3.5.4.1 Normal distribution

Also known as the Gauss distribution, the normal distribution is seen as the most important type of a continuous random variable denoted as $N(\mu, \sigma)$. Dependent only on the mean and standard deviation, the normal distribution is often used as a useful approximation of more complicated distributions. A normal distribution is symmetric with respect to its mean. The normal distribution is convenient to use for a random variable with a coefficient of variation



smaller than 0.3. The probability density function of a normal distribution is given by Equation 3.11 (Lungu, 2007; Holicky, 2009).

$$\varphi(x) = \frac{1}{\sigma_X \sqrt{2\pi}} \exp\left[-\frac{1}{2} \left(\frac{x - \mu_X}{\sigma_X}\right)^2\right] \quad (3.11)$$

As discussed in the previous section, the random variable is often standardised to ease the process of determining probabilities. By substituting the mean and standard deviation with zero and one respectively, the probability density function of a standardised normal distribution can be expressed as

$$\varphi(u) = \frac{1}{\sqrt{2\pi}} \exp\left[-\frac{u^2}{2}\right] \quad (3.12)$$

3.5.4.2 Log-normal distribution

The unlimited interval of a normal distribution is undesirable in some practical applications. The log-normal distribution partly eliminates this as it is asymmetric on a one-sided limited interval $x_0 < x < \infty$ or $-\infty < x < x_0$. Positive asymmetry occurs when the distribution is shifted to the left. A random variable X has a log-normal distribution if the random variable Y has a normal distribution (Holicky, 2009).

$$Y = \ln |X - x_0| \quad (3.13)$$

Depending on the mean, standard deviation and the skewness, it is often referred to as the three-parameter log-normal distribution. When the skewness is uncertain or unknown the upper or lower bounds x_0 may be used. When a log-normal distribution has a lower bound of zero, it is referred to as a two-parameter log-normal distribution. This is the most frequently used log-normal distribution and is generally used for resistance properties of materials (Lungu, 2007). The probability density function of a log-normal distribution for a standardised variable is expressed as

$$\varphi_{LN,U}(u) = \frac{\varphi(u')}{\left(u + \frac{1}{c}\right) \sqrt{\ln(1+c^2)}} \quad (3.14)$$

3.5.4.3 Gamma distribution

The gamma distribution is another example of a one-sided limited distribution with the lower bound equal to zero. Although both the gamma and the log-normal distributions have lower bounds of zero, they differ in skewness where the skewness of the gamma distribution is much lower. The gamma distribution is mostly used to describe geometrical quantities and action effects that do not have a great skewness (Holicky, 2009).



3.5.4.4 Beta distribution

Such as the normal distribution, the beta distribution is defined on a two-sided interval, but unlike the normal distribution the interval is limited to upper and lower bounds. Due to the fact that the beta distribution depends on four parameters, it is difficult to use such a distribution in a practical application (Holicky, 2009).

3.5.4.5 Gumbel and other distributions of extreme values

The distribution of the extreme values in a population is important to model loads such as wind and wave actions. The most frequently used extreme value distribution is the Gumbel distribution which only depends on the mode x_{mod} and the parameter $c > 0$. These type distributions have two versions, one for the minimum values and one for the maximum values. Defined on an infinite interval the Gumbel probability density function has the form

$$\varphi(x) = c \exp(-c(x - x_{mod}) - \exp(-c(x - x_{mod}))) \quad (3.15)$$

where $x_{mod} = \mu - 0.577\sigma \frac{\sqrt{6}}{\pi}$, $c = \frac{\pi}{\sigma\sqrt{6}}$. The skewness and kurtosis of the Gumbel distribution are taken as 1.14 and 2.4 respectively (Holicky, 2009).

3.5.4.6 Choosing the most appropriate distribution

The normal distribution may seem like the most applicable and useful type of distribution, but when it comes to asymmetric variables the normal distribution may deliver inaccurate results as it does not take the skewness into account. Therefore, it is important to be able to identify and use the appropriate distribution for a given set of experimental data. The display of histograms can provide a good insight about possible distributions to use as a model to represent the data. When large populations of data are used, a histogram can provide a reasonable indicator of the shape of the distribution of the data (Montgomery, 2003).

Normal distributions are frequently used to represent various types of random variable describing loads, geometrical and mechanical properties. Normal distributions work best for a symmetric random variable with a coefficient of variation smaller than 0.3 and a skewness smaller than or equal to 0.5. When working with data that is not symmetrically distributed, an asymmetric (Log-normal) distribution can be used (Holicky, 2009).

Another way of determining what distribution will fit a set of data the best is by performing a Goodness of Fit test (GoF). The Chi-Square GoF test is an example of such a test. This test is a useful technique for testing whether observed data are representative of a particular distribution. In other words, a distribution is assumed to adequately represent an observed set



of data and then by performing the GOF test, the hypothetical distribution is either accepted or rejected.

By using the observed and expected values, the Chi-square tests statistic is calculated using Equation 3.16. The result of this is compared to the corresponding Chi-squared percentage point. Dependent on the sample size, degrees of freedom and the chosen confidence interval (usually 95%) the percentage points are derived from published tables. The Chi-square percentage point is denoted as $\chi^2_{\alpha,v}$ and the test statistic as χ^2 . The hypothetical distribution is rejected when $\chi^2 > \chi^2_{\alpha,v}$. This is shown in Figure 3.4.

$$\chi^2 = \sum \frac{(\text{Observed} - \text{Expected})^2}{\text{Expected}} \quad (3.16)$$

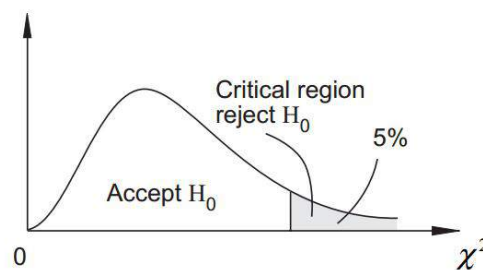


Figure 3.4 Chi-Squared distribution

3.5.5 The First Order Reliability Method (FORM)

The final step in a reliability analysis is the determination of the reliability measures. Generally, the methods used to determine the reliability status of a structure can be categorised in four different levels. The classification is done based on what the uncertain parameters are modelled with in order to measure the reliability (Sorensen, 2004).

- Level 1: Uncertain parameters modelled by one characteristic value
- Level 2: Modelled by means and standard deviations.
- Level 3: Modelled by the joint distribution functions of uncertain quantities.
- Level 4: The risk is used as a reliability measure and the cost of failure is accounted for.

Depending on the required level of accuracy, several techniques are available to estimate the reliability index and corresponding probability of failure. Level three is considered for this research and one of the techniques that are often used to estimate the reliability for the methods of this level is the First Order Reliability Method (FORM) (Sorensen, 2004). This method is one of the most basic and efficient reliability methods and is regularly used in software products to determine the reliability of structures.



Before discussing the procedure of a typical FORM analysis, it is important to first discuss some of the parameters that are used in and derived from such an analysis. In support of the discussions, Figure 3.5 gives a graphical illustration of the different parameters. The horizontal axis represents the structural resistance where the action effect is shown on the vertical axis. By assuming that variables R and E are mutually independent and that both have normal distributions, then the joint probability distribution function can be represented by the concentric circles. These circles are corresponding to the different levels the probability density.

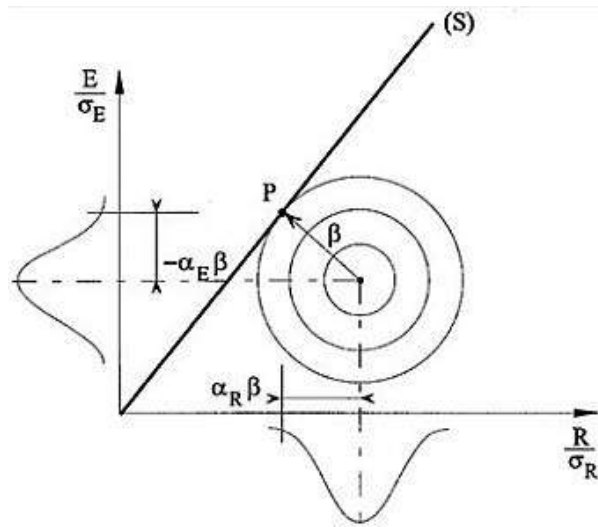


Figure 3.5 First order reliability method

The first parameter worth mentioning is the design point which is denoted as P in Figure 3.5. The design point is the point on the limit state function (S) that has the smallest reliability index and highest probability of failure. The distance between the design point and the origin of the joint probability distribution is denoted by β (reliability index) and can be determined using

$$\beta = \frac{\mu_R - \mu_E}{\sqrt{\sigma_E^2 + \sigma_R^2}} \quad (3.17)$$

where μ_R and μ_E are the means of the respective variables. These two points can also be used to determine the design point using Equations 3.18 and 3.19. The alpha values in Figure 3.5 are known as the FORM sensitivity factors and are calculated using Equations 3.20 and 3.21.

$$e_d = \mu_E - \alpha_E \beta \sigma_E \quad (3.18)$$

$$r_d = \mu_R - \alpha_R \beta \sigma_R \quad (3.19)$$

$$\alpha_E = \frac{-\sigma_E}{\sqrt{\sigma_E^2 + \sigma_R^2}} \quad (3.20)$$



$$\alpha_R = \frac{\sigma_R}{\sqrt{\sigma_E^2 + \sigma_R^2}} \quad (3.21)$$

The FORM sensitivity factors can be used as an indication of each variable's contribution towards the reliability of the structure. The variable that has the highest alpha value contributes the most towards the reliability. The procedure of a typical FORM analysis can further be summarised by the following points.

1. The limit state function $G(x) = 0$ is defined and the variables $X = \{X_1, X_2, \dots, X_n\}$ are identified.
2. An initial design point $x^* = \{x_1^*, x_2^*, \dots, x_n^*\}$ is calculated by using the limit state function and the mean values of $n - 1$ variables.
3. The equivalent normal distributions are found for all variables, $U = \{U_1, U_2, \dots, U_n\}$.
4. The transformed design point $u^* = \{u_1^*, u_2^*, \dots, u_n^*\}$ is determined using Equation 3.22.

$$u_i^* = \frac{x_i^* - \mu_{X_i}^e}{\sigma_{X_i}^e} \quad (3.22)$$

5. The limit state function is derived with respect to each variable after which the derivatives is presented by vector D .

$$D = \begin{bmatrix} D_1 \\ D_2 \\ \vdots \\ D_n \end{bmatrix} \text{ where } D_i = \frac{\partial G}{\partial U_i} = \frac{\partial G}{\partial X_i} \frac{\partial X_i}{\partial U_i} = \frac{\partial G}{\partial X_i} \sigma_{X_i}^e \quad (3.23)$$

For a linear limit state function $a_0 + \sum a_i X_i = 0$ the derivatives are $D_i = a_i$

6. By using Equation 3.24 the reliability index β is then calculated.

$$\beta = \frac{a_0 + \sum a_i \mu_{X_i}^e}{\sqrt{\sum (a_i \sigma_{X_i}^e)^2}} \quad (3.24)$$

7. The vector of sensitivity factors $\{\alpha\}$ are determined as

$$\{\alpha\} = \frac{\{D\}}{\sqrt{\{D\}^T \{D\}}} \quad (3.25)$$

8. The sensitivity factors and reliability index are used to determine new design point.

$$u_i^* = \alpha_i \beta \quad (3.26)$$

$$x_i^* = \mu_{X_i}^e - u_i^* \sigma_{X_i}^e \quad (3.27)$$



9. The design point value of the remaining variables is then determined using the limit state functions.
10. Steps three to nine are repeated until β and the design point has the required level of accuracy.

3.6 Concluding remarks

The main objective of the current research is to report on the reliability of screwed connections in tilt-and bearing. The chosen method of analysing these connections is based on probabilistic techniques. Also known as the reliability theory, all identifiable uncertainties are rationally assessed during such a probabilistic approach.

A reliability analysis starts of by identifying the significant failure mode and then formulating a performance function of this failure mode. Once all variables and parameters in this function are identified, appropriate distribution types and statistical parameters are specified to represent these variables. A First Order Reliability Method (FORM) is used to estimate the reliability index, sensitivity factors and the design point with the last two used to describe the respective variables' role and contribution towards the overall resistance reliability. The reliability index is used to determine the reliability status. In order for the connections to be declared reliable, the computed reliability index must meet the specified target reliability index.

Chapter 4

Research Methodology

4.1 Introduction

This chapter provides information on the fundamental research methods used in determining and investigating the resistance reliability of typical cold-formed steel (CFS) screwed connections. These research methods include experimental investigations, statistical inference and reliability analyses.

Typical CFS screwed connections were identified as connections consisting of extremely thin-walled elements that typically fail in screw tilting and hole bearing. The reliability of these connections were determined through an analysis where the probability of achieving an actual capacity less than the SANS design capacity was evaluated. The design capacity was determined through a deterministic application of the SANS formulation where a probabilistic approach was followed for the actual capacity as it needs to account for various uncertainties, including parameter randomness and model uncertainty.

4.2 Typical failure mode

A typical screwed connection in CFS structures consists of two or more extremely thin-walled CFS members connected by a single or multiple screw(s). CFS screwed connections are designed against shear failure of the screw(s), end tear-out, net-section failure and tilt-and bearing failure. For each of these failure modes, the SANS 10162-2 provide a corresponding capacity prediction equation which is a function of numerous material and geometrical properties. The first step of the current investigation was to identify the failure mode that would typically occur when using materials generally used in the South African engineering practice. According to the director of the Southern African Light Steel Frame Building Association (SASFA), John Barnard,



the properties of CFS most frequently used for light steel frame construction in SA is as shown in Table 4.1.

Table 4.1 Tested steel nominal properties

Steel property set	Steel grade (MPa)	Steel thickness (mm)	Coating (g/m ²)
1	550	0.8	Z275
2	550	0.58	Z275
3	300	1.0	Z275
4	230	0.8	Z275

In general, screw failure and end tear-out failure can easily be prevented by using specific geometry and screw properties. According to the SANS 10162-2, section tear-out will not occur when the end distance is at least equal to 1.5 times the nominal screw diameter. When using a screw with a nominal shear capacity of 1.25 times the nominal bearing capacity, the screw will not fail in shear. The remaining two failure modes cannot as easily be prevented and manipulated and therefore various literature was studied in order to identify the expected failure mode.

Just to name a typical study of such, Rogers and Hancock (1999) did a study on the behaviour of single screwed connections subjected to shear. The same as all other similar studies, they found that single screwed connections are most likely to fail in a combination of screw tilting and hole bearing. This failure mode was said to be caused by the extreme thinness of the connected members that induces an eccentric shear load on the screw, causing it to tilt. The steel thicknesses considered here (Table 4.1) do not exceed 1.0 mm and can therefore be categorised as extremely thin. It can thus be concluded that the tilt-and bearing failure mode is expected to occur when applying a shear force to single-point screwed connections with steel properties as in Table 4.1.

4.3 Resistance prediction model

The SANS 10162-2 provides three equations for designing screwed connections with equally thick members against the combined failure mode of screw tilting and hole bearing. In reference to Section 2.7.3, two of these equations represent a failure mode where bearing is the main contributor towards failure. The third equation represents tilting-and bearing failure where tilting is dominant. Due to the extreme thinness of the materials, as discussed in Section 4.2, the connections considered here are expected to fail primarily due to tilting. Equation 4.1 gives the



nominal capacity for screwed connections with equally thick members expected to fail in screw tilting and hole bearing, with tilting the dominant contributor towards failure, according to SANS 10162-2.

$$V_b = 4.2 \sqrt{t^3 d_f f_u} \quad (4.1)$$

where t is the thickness of the connected member not in contact with the screw head, d_f the nominal screw diameter and f_u the ultimate tensile strength of the connected members. By excluding conservative parameter bias as well as partial-, safety- and capacity reduction factors, Equation 4.1 was used as the resistance prediction model for predicting typical screw connection capacities. By using measured parameter values instead of nominal values, this prediction model delivered unbiased connection capacity predictions that were used for the model factor estimation.

4.4 Uncertainties

Although the resistance prediction model delivers unbiased capacity estimates, these estimates will still deviate from the true capacity due to numerous uncertainties. More often than not, the true value of a material property is different than the nominal value used in design. This natural randomness of materials causes variable (material and geometrical) uncertainties. During a reliability analysis (Section 4.5), the uncertainties associated with these variables are probabilistically assessed. Characterised by a mean and standard deviation, each of the respective variables are modelled using an appropriate statistical distribution. Steel properties, such as the tensile strength, are usually modelled by a log-normal distribution with a coefficient of variation (standard deviation/mean) between 7% and 10%. Normal distributions are used for modelling geometric variables (steel thickness, screw diameter) and usually have a coefficient of variation between 1% and 4%.

Other than the variable uncertainties, there also exist uncertainties within the model itself. These uncertainties need to be considered and quantified which is usually done in the form of a model factor by comparison with collected data from experimental work.

4.4.1 Model factor determination

By using actual measured values of the true material and geometrical properties for a capacity prediction, the uncertainties associated with these material and geometrical variables are excluded from those causing the predicted capacity to deviate from the actual capacity. The remaining uncertainties can be quantified in the form of a ratio between the actual (tested)



capacity and the capacity predicted, also known as the model factor. These uncertainties include those directly associated with the prediction model, experimental design and the test set-up as discussed in Section 3.3.3. The model factor is defined as the ratio of actual connection capacity over the unbiased predicted capacity.

$$\theta = \frac{V_{actual}}{V_{predicted}} \quad (4.2)$$

The actual connection capacity is determined by testing, with the corresponding unbiased prediction based on Equation 4.1, but calculated using measured parameter values instead of nominal (biased) values. To this end experimental work was necessary: Measurements were made of the plate thicknesses and screw diameters and tensile tests were performed to obtain the tensile strength used in the capacity predictions. In addition, connection capacity tests were performed to obtain the actual connection capacities. The tests are discussed in more detail in Sections 4.4.1.2 and 4.4.1.3.

A model factor was determined for each of the connection specimens and by interpreting each of these individual model factor values as realisations of the overall model factor, the model factor was seen as a continuous random variable. Through a probabilistic distribution of this random variable, the necessary statistical parameters was obtained and used to describe the model factor. The average model factor indicated whether the resistance prediction model is conservative or not and the standard deviation indicated the model's level of uncertainty. An average model factor less than one, implies a model that tends to over predict the capacity while an average model factor exceeding one implies predictions that tends to be conservative. The model factor determination is thoroughly discussed in Chapter 6.

4.4.1.2 Connection tests

In Appendix F of the SANS 10162-2 a standard test for single-point fastener connections are proposed for evaluating the structural performance of screwed connections. This proposed method of testing was used for the determination of typical screw connection capacities. Consisting of two equally sized steel strips, connected with a single screw, the connections were subjected to a shear force until the specified limits were reached. A total of 222 screw connections and an additional 15 riveted connections were tested. Together with the predicted capacities, these tested capacities were used to determine model factors. The experimental design and results of the connection tests are presented in Chapter 5.

4.4.1.2 Capacity predictions

By using the appropriate nominal resistance model (Equation 4.1) in SANS 10162-2, the unbiased capacity of each specimen was predicted. The resistance prediction model is a



function of the steel thickness, fastener nominal diameter and steel strength of which all three these variables had to be obtained for each of the tested specimens. By using actual measured geometric and material properties for the capacity predictions, the uncertainties directly associated with these variables are excluded from those causing the predicted capacity to deviate from the actual capacity. The screw diameter and steel thickness were physically measured with the tensile strength obtained through tensile testing. The capacity predictions form part of the model factor determination which is presented in Chapter 6.

All tensile tests were done according to the specifications of SANS 6892-1:2010 (Tensile testing of Metallic materials-Method of test at room temperature). The main objective of the tensile tests was to determine the ultimate tensile strength of the respective steel grades (Table 4.1). The strength variation within a single steel coil was found to be small enough so that the average of three measurements was used as the measured ultimate tensile strength of a coil. The experimental design and results of the tensile tests are presented in Chapter 5.

4.5 Reliability analysis

A reliability analysis can be defined as a probabilistic approach in determining the reliability status of a structure or structural component. Such an analysis is an iterative process where the most likely point of failure and its corresponding failure likelihood is determined. Through the use of a First Order Reliability Method (FORM), a given Limit State Function (LSF) is analysed, resulting in a computed reliability index. This computed value must meet the specified target value in order for the connection to be declared reliable.

Any structure has a state where it is safe and a state where it is unsafe. The limit between these two states can mathematically be expressed by a limit state function (LSF). The LSF for the current analysis was defined as the difference between the actual connection capacity and the design capacity. When the actual capacity of a connection is less than the capacity designed for, the connection will fall on the unsafe side of the LSF.

As the thickness, screw diameter and tensile strength were determined for each of the specimens during the capacity predictions, these sample characteristics can be put together as random variables for the respective input parameters. By representing each of the parameters in the resistance prediction model with the appropriate probability distributions and by multiplying the model with the model factor (probabilistic) the resulting expression is an estimate of the actual capacity. As a probabilistic model, this term consists of numerous variable distributions taking the corresponding uncertainties into account. To complete the LSF, the



design capacity was computed deterministically based on the design provisions of SANS 10162-2. The LSF is thus defined as

$$G = \theta_{(\mu, \sigma)} * 4.2 \sqrt{t_{(\mu, \sigma)}^3 d_{f, (\mu, \sigma)} f_{u, (\mu, \sigma)} - V_d} \quad (4.3)$$

Reliability analyses was performed for screwed connections using the CFS properties in Table 4.1 and also for some riveted connections using the 0.8 mm, 550 MPa steel. Chapter 7 discusses these reliability analyses and results.

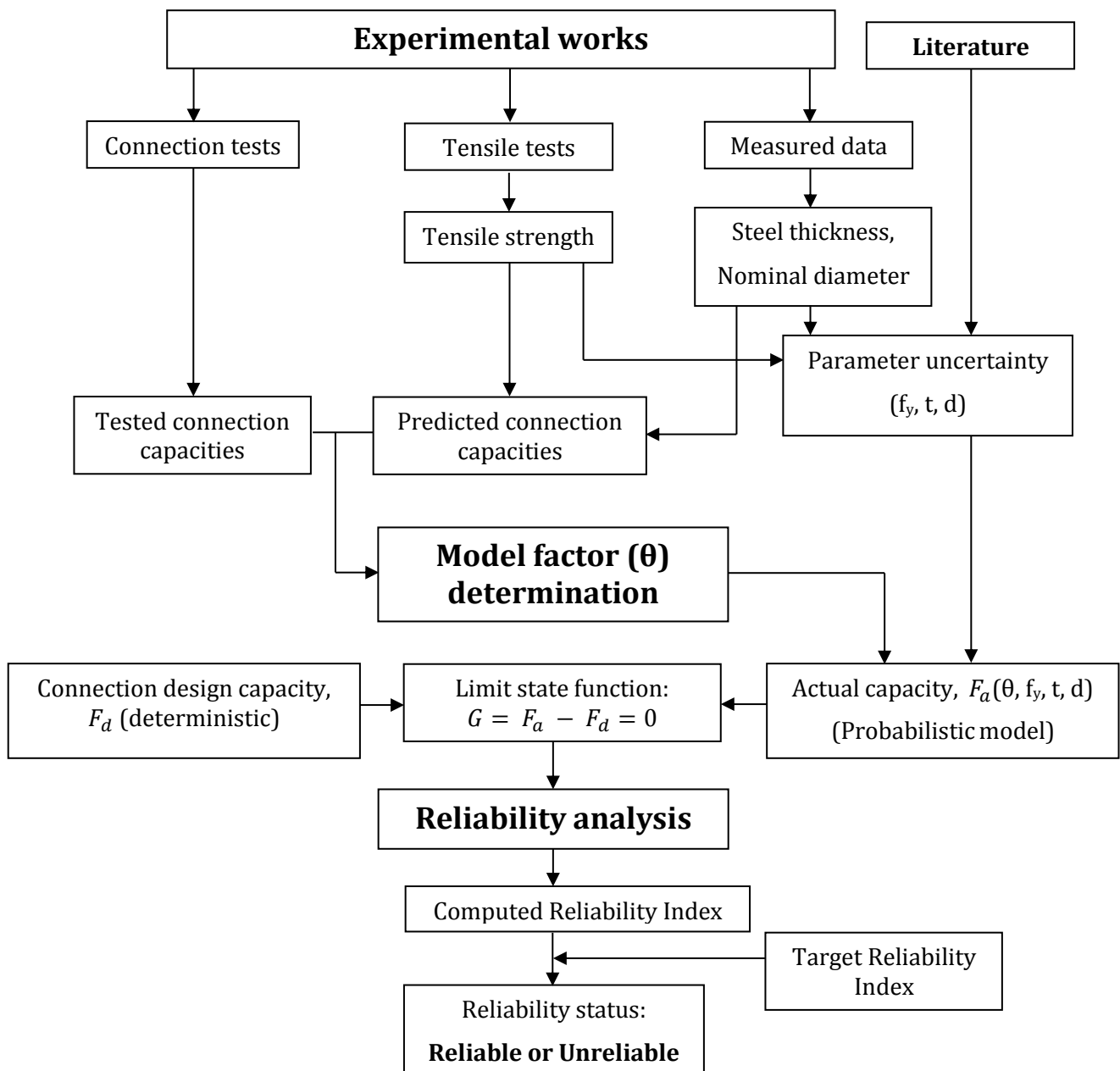


Figure 4.1 Research methodology diagram

Chapter 5

Experimental Design and Results

5.1 Introduction

Experimental work is required to determine a model factor, as outlined in Chapter 4. This chapter will discuss the different test procedures, measurements made and the experimental results. To recap, a model factor is defined as the ratio between the predicted and actual connection capacities. In order to make unbiased capacity predictions, measured values of the steel strength (Section 5.2), plate thickness (Section 5.3) and screw diameter (Section 5.3) were needed, for each specimen. The actual connection capacities were determined through testing (Section 5.3). Using various South African national standards, the experiments were designed to meet all necessary specifications regarding the test configuration, test specimen and materials used.

A model factor was determined for each of five different typical connections as shown in Table 5.2. These connections can be differentiated mainly based on their steel property (strength and thickness) set and connector type (screwed or riveted). The four sets of properties are shown in Table 5.1 together with the amount of steel coils used of each set.

Table 5.1 CFS categories used for experimental work

Steel property set	Steel grade (MPa)	Steel thickness (mm)	Coating (g/m ²)	Number of coils tested
1	550	0.8	Z275	4
2	550	0.58	Z275	1
3	300	1.0	Z275	1
4	230	0.8	Z275	1



Table 5.2 Typical connection configuration

Typical connection	Steel property set	Connector type	Number of specimens tested
1	1	screwed	132
2	2	screwed	30
3	3	screwed	30
4	4	screwed	30
5	1	riveted	15

5.2 Tensile testing

It was impossible to test the strength of the exact specimen that still needed to be used in the capacity tests, due to the destructive nature of the tensile tests. For each of the steel grades, the average result of three tensile tests, within a single steel coil, were found to be adequately representative of the tensile strength of that particular coil. This average coil tensile strength was used as the tensile strength in the resistance prediction model for capacity predictions of connections with members originating from the same coil. This was done for the three different steel grades used. To ensure trustworthy results, the tensile tests were strictly executed according to various specifications given in different SANS design codes as set out below.

5.2.1 Test specifications

SANS 10162-2 suggests that any tests of CFS members in order to determine any sort of physical property be carried out under the conditions specified in the SANS 6892. Also specified in SANS 10162-2 is that a specimen must be cut from a coil so that the long dimension of the specimen is in the rolling direction and at a position located one quarter of the coil width from either edge near the outer end of the coil (SABS, 2011).

5.2.1.1 SANS 6892

Tensile tests for the determination of yield and tensile strengths must take place at room temperatures between 10°C and 35°C. The testing machine must have clamps/grips that will not allow the specimen to slip during testing. Preferred test specimens have a direct relationship between the gauge length (L_o) and the cross sectional area (S_o) expressed by the equation $L_o = k\sqrt{S_o}$ where k is a coefficient of proportionality. The internationally adopted value for k is 5.65. A gauge length of more than 20 mm is proposed as a smaller length is said to increase measurement uncertainty. Referring to Figure 5.1, the following physical



characteristics are given in ANNEX B of the SANS 6892-1 regarding tests pieces to be used for thin products between 0.1 mm and 3 mm thick.

- The ends of the specimen must be connected to the parallel length via transition curves with a minimum radius of 20 mm.
- The width of the ends should be $\geq 1.2b_o$
- The parallel length (L_c) shall not be less than $L_o + b_o/2$.

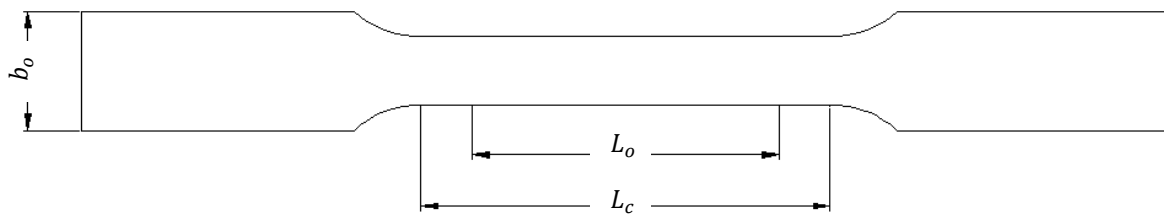


Figure 5.1 Standard test specimen

In general, there are three sets of specimen geometries used (Table 5.3) with test piece type 2 used in this experiment. In order to achieve results with minimum measurement uncertainty it is recommended that the original cross sectional area be determined with an accuracy of $\pm 1\%$. If it is only the tensile strength and the lower yield strength of the material that are being tested, the rate at which the specimen is strained (strain rate) can be taken as 0.002 mm per parallel length per second.

Table 5.3 Specimen geometries

Test piece type	Width b_o (mm)	Original gauge length L_o (mm)	Parallel length L_c (mm)		Free length between grips (mm)
			Minimum	Recommended	
1	12.5 ± 1	50	57	75	87.5
2	20 ± 1	80	90	120	140
3	25 ± 1	50	60	-	Not defined

5.2.2 Test specimen

5.2.2.1 Specimen material

The steel types for which the SANS 10162-2 is applicable is discussed in Section 2.7.2. All of these requirements are met by the CFS most frequently used in the South African practice. The properties of this CFS are listed below.

- Steel grade: ISQ 550, ISQ 300 or ISQ 230 (Commercial Quality).
- Steel thickness: 0.58, 0.80 or 1.0 mm.
- Coating: Zinc coating – mainly Z275, also Z200 (AZ150 or AZ100).



The CFS that is used the most is has a 550 MPa steel grade with a Z275 coating and a thickness of 0.8 mm. This was the primary steel used throughout the experiment. Other than the 550 MPa, 0.8 mm steel, all of the above listed strengths and thicknesses were used for producing specimens. The combinations in which these properties were used are given in Table 5.1.

5.2.2.2 Specimen preparation

The tensile test specimens were either cut from a 1225 mm or a 182 mm wide coil. Obliging to the specifications discussed in Section 5.2.1 each of the specimen were cut from the coil in the rolling direction and at a position located one quarter of the coil width from either edge near the outer end of the coil. Figure 5.2 shows the position of a specimen cut from a 182 mm coil. To avoid localised hardening or any other influence on the material properties, water jet cutting was used to cut the specimens from the steel coils.

A diluted solution of hydrochloric acid was used to remove the coating of the specimens in order to obtain the base metal thickness (BMT). Five measurement locations and two guidelines were marked of on the specimens. The BMT was measured at each of these measurements locations of which an average value was determined. To ensure that the specimens were clamped in at a right angle and that the force was applied uniformly, the guidelines were paralleled with the edges of the clamp ends. To prevent rusting from influencing the test results, the specimens were immediately tested after the measurements took place. Evident in Figure 5.3 is the removed coating, the five measurement locations and the two guidelines.

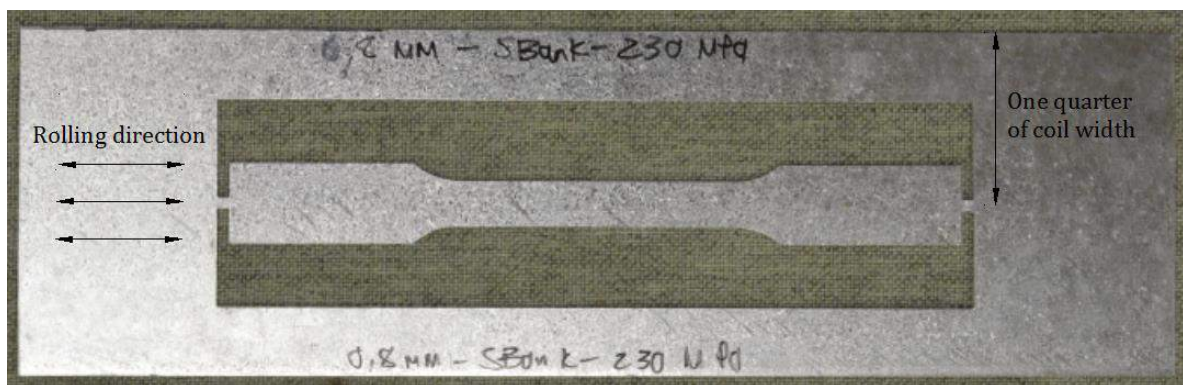


Figure 5.2 Location of specimen within coil

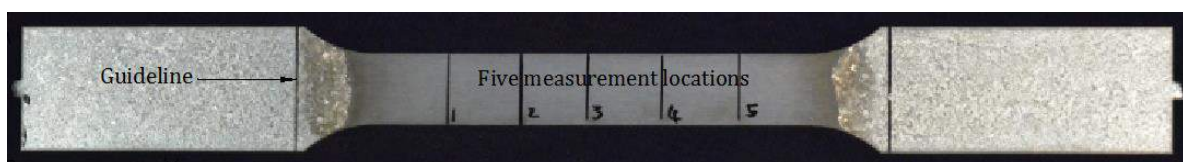


Figure 5.3 Prepared tensile test specimen



5.2.2.3 Specimen geometry

Figure 5.4 illustrates the geometry of a tensile test specimen. The dimensions shown in Figure 5.4 remained the same for all plate thicknesses.

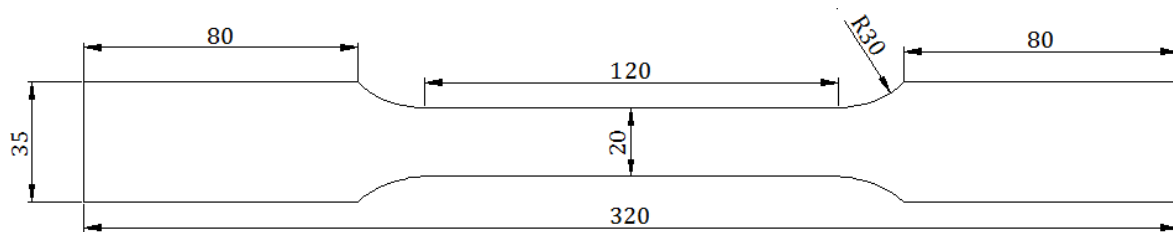


Figure 5.4 Tensile test specimen geometry (measurements in mm)

5.2.3 Measurement instrumentation

All tests were done in the structures laboratory of the engineering faculty at the University of Stellenbosch. The machine used for the tensile tests were the Zwick/Z250, of which an example is shown in Figure 5.5. Consisting of two platforms (heads) the strain rate and thus also the specimen elongation was controlled and measured using the crosshead separation. Another more accurate method of controlling and measuring the specimen elongation is by using a displacement transducer or a Linear Variable Differential Transformer (LVDT) over a specified gauge length. This method is used when strain is to be determined. The SANS 6892 however states that the crosshead separation elongation measurement is sufficient if the only purpose of the test is to determine the ultimate tensile strength. The ultimate tensile strength was the only property to be obtained from the tensile tests of the current experimental investigation. The reason why it is not acceptable to determine strain and thus stiffness with the crosshead separation is that the cross section between crossheads varies. The strain and stiffness was not required at any stage during this thesis and therefore it was not necessary for any gauge length measurements.

The load was measured with a load cell connected to the Zwick. The Zwick was connected to a personal computer on which the specimen extension and the tensile force were recorded. The upper platform is fixed while the lower platform moves relative to the upper platform applying either a tensile or a compression force, depending on the direction. Situated on each of these platforms is a clamping system that operates hydraulically. The pressure is regulated through a gauge and the clamps are opened and closed using a hydraulic switch, connected to an air compressor. Other instruments used during the experimental stage were a micro meter and digital calliper. These were used to measure the steel thicknesses and specimen widths respectively.



Figure 5.5 Zwick/Z250 machine

5.2.4 Test execution methodology

Using the horizontal and vertical lines drawn on the specimens as reference, the specimen were inserted and clamped into the Zwick. This alignment was important to ensure that the load was applied uniformly and that no eccentricity was present. As a slight force is automatically recorded whenever the lower platform is moved, the recorded force was always zeroed before the test started. Applying a tensile force to the clamped-in specimen, the lower platform was set to move downwards at a rate of 0.024 mm per second. The connected computer recorded the results in the form of a graph, with the tensile load on the Y-axis and the displacement on the X-axis. The test specimen was loaded until total fracture of the specimen's cross-section occurred. Figure 5.6 shows an original and fractured clamped-in specimen respectively.



Figure 5.6 Original specimen (left) and fractured specimen (right)

5.2.5 Test data analysis

The data for the tensile tests was analysed using the force-displacement curves plotted by the computer that were connected to the Zwick. The force-displacement curves were converted to stress-strain curves, using the original cross-sectional area and length of the specimens. Figure 5.7 represents a typical stress strain curve. The data analysis included the reading off of the material's yield strength (lower yield strength) and its tensile strength at the locations shown in Figure 5.7.

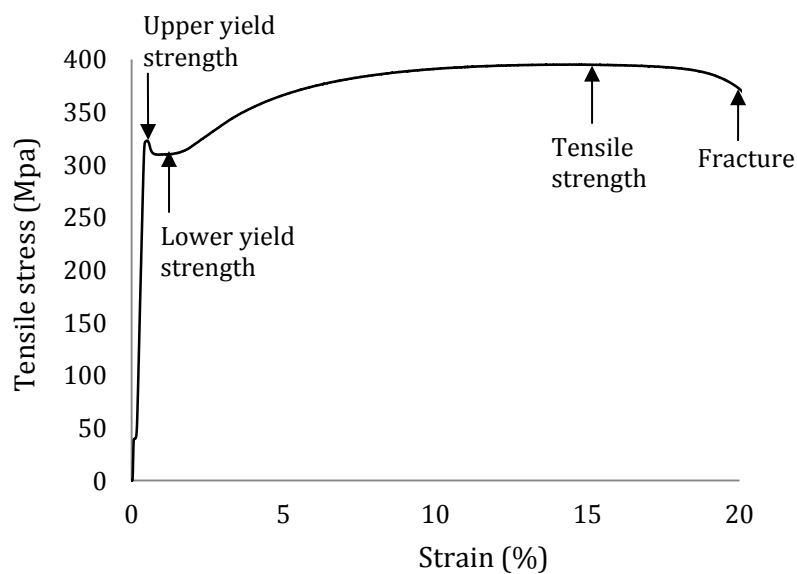


Figure 5.7 Stress-strain curve



5.3 Connection capacity testing

5.3.1 Test specifications

In APPENDIX F of the SANS 10162-2 a standard test for single point fastener connections are given. This proposed method and its specifications formed the basis of the connection capacity tests performed.

5.3.1.1 SANS 10162-2 (APPENDIX F)

This standard test is applicable to specimens consisting of two strips of steel sheets, connected by a single fastener through overlapped ends. The strips must be joined flat together and free of any residue with the fastener installed within 3.0 mm of its specified location. Figure 5.8 together with Table 5.4 shows the recommended specimen geometry.

The testing apparatus used must have clamps capable of holding the ends of a test specimen in such a way as to ensure uniform loading. When the thickness at each end exceeds 2.0mm, packing shims or adjustable grips must be used to ensure central loading across the lap joint. The specimen has to be loaded at a controlled rate and must be completed within a 30 s to 240 s time frame. The test must be stopped once the maximum load has been reached and the load has either dropped off or the joint has undergone a displacement of 6.0 mm.

Table 5.4 Specified connection specimen geometry

Fastener	Width of specimen w (mm)	Lap length l_a (mm)	Gauge length for measuring joint displacement l_g		Unclamped length of specimen l_c (mm)
			Minimum (mm)	Maximum (mm)	
Clinches and all other fasteners with shank diameters ≤ 7.0 mm	50	50	100	150	150
All fasteners with shank diameters > 7.0 mm	8*shank diameter (d_{sh})	$8d_{sh}$	$16d_{sh}$	$24d_{sh}$	$24d_{sh}$

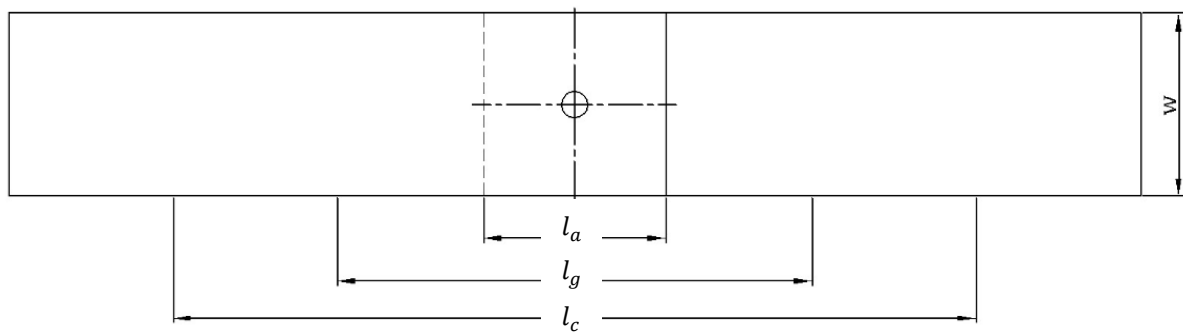


Figure 5.8 Specified connection test specimen

5.3.2 Test specimen

5.3.2.1 Specimen material

As the tensile tests were done to determine the steel strength of the steel strips used for the connection tests, the steel specifications discussed under Section 5.2.2.1 also applies here. All that is added is the fasteners used to assemble the connection specimens. A total of 237 connections was tested of which 15 were pop-riveted connections and the remaining 222 all consisted of two steel strips fastened with a 4.8 mm wafer head self-tapping screw. There are not a lot of different diameters used for the purpose of CFS structures in South Africa as the allowable screw diameter range is only between 3 mm and 7 mm (SANS 10162-2). After visiting numerous construction sites and light steel frame housing factories, the screw diameter that was found to be used exclusively, was the 4.8 mm diameter. This was the argument for only testing 4.8 mm connectors. However, to be academically thorough, the range should be extended to cover what is allowed by the code. This is further discussed in Chapter 8 (recommendations for future work). Typical 4.8 mm pop rivets and 4.8 mm wafer head self-tapping screws are shown in Figure 5.9.



Figure 5.9 Pop-rivets and screws used for connection specimens



5.3.2.2 Specimen preparation

Steel is anisotropic and was therefore important to stay consistent with the direction in which the specimens were cut from the coils. The steel strips used for the connection specimens were cut from the coils with their length parallel to the rolling direction. Using a hydraulic guillotine, the strips were cut according to the dimensions discussed in Section 5.3.2.3. After the required lap length was marked off, the steel strips were clamped on top of one another. Pilot holes were first drilled after which the strips were fastened together using a wireless screw driver. The pop rivets were inserted with a pop rivet gun. Examples of assembled screwed connections are shown in Figure 5.10. Any residue was removed to ensure that the strips were joined flat together.



Figure 5.10 Assembled screwed connection specimens

Once a test was completed, the galvanised coating was removed with the use of a diluted hydrochloric acid. Measured at the four corners within the lap length an average BMT for each individual strip was obtained. These measurements are given in Section 5.6 and were used for the capacity predictions in Chapter 6. The removed coating is evident in Figure 5.11 together with the four measurement locations.

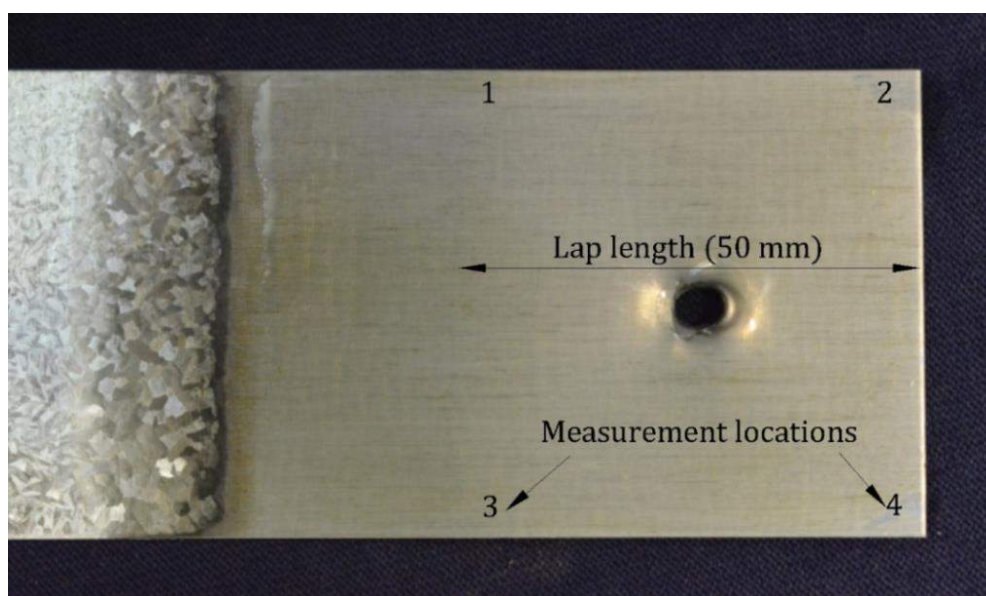


Figure 5.11 Removed coating and thickness measurement locations



5.3.2.3 Specimen geometry

The connection capacity specimens consisted of two identical steel strips joined together using a single screw (or pop-rivet). Although the thickness varied among specimens, the other dimensions stayed the same and are shown in Figure 5.12.

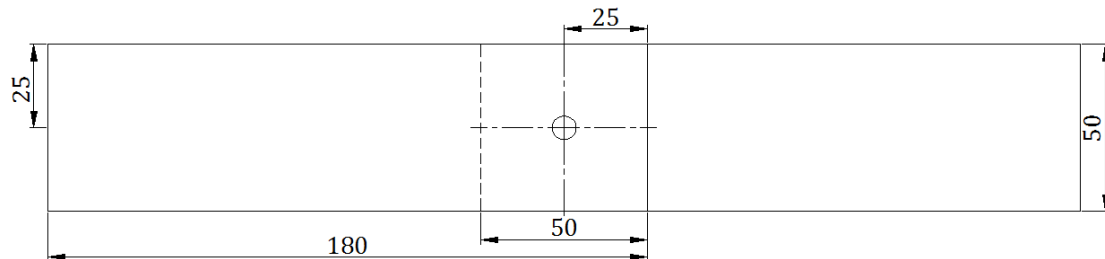


Figure 5.12 Connection specimen geometry

5.3.3 Measurement instrumentation

The same machine (Zwick/Z250) was used for the connection capacity tests as for the tensile tests. The description of the measurement instrumentation discussed under 5.2.3 therefore also applies here. A digital calliper and micro meter was used to measure the dimensions of the connection specimens. Not being able to accurately measure the nominal screw diameters with either of these measurement instruments a different method was used. Using a microlensed camera, photos were taken of each of the screws and then with the use of Computer Aided Design (CAD) the diameters were measured. By using a scale, the ratio of the known distance between a calliper's claws and the equivalent distance in the imported CAD image and the CAD diameter of the screw, the true diameters were obtained. The true (measured) diameters are given in Section 5.6 and were used for the capacity predictions in Chapter 6. An example of a CAD image used in determining the screw diameters is shown in Figure 5.13.

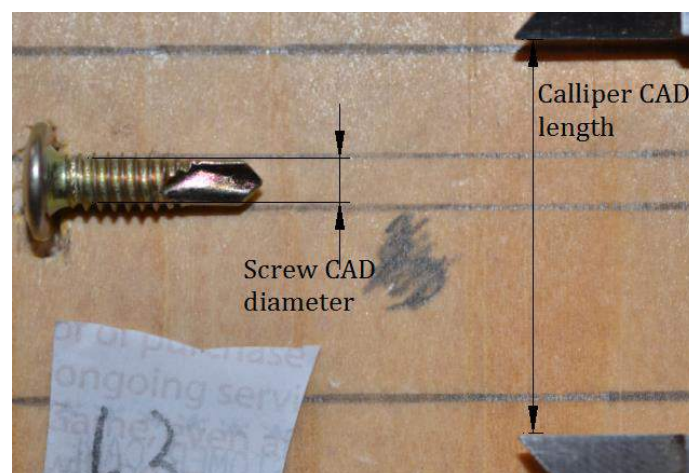


Figure 5.13 Nominal screw measurement



5.3.4 Test execution methodology

Although being specified as only necessary when the ends exceed a thickness of 2 mm, it was decided to use end plates to eliminate eccentricity and to ensure that the load was applied through the centre of the specimen. On both the specimen and the end plates, guidelines were drawn that were used for alignment. A clamped-in connection specimen prior to testing is shown in Figure 5.14. The rate at which the force was applied was determined using the allowable test duration and specimen displacement. To allow the specimen to reach the specified displacement of 6 mm within a time frame of 30 s to 240 s, the strain rate was chosen at 0.03 mm per second. The test was stopped at a crosshead separation of about 8 mm.

5.3.5 Test data analysis

Most of the tests were only stopped at a crosshead separation of around 8 mm. Even though this is past the point where the SANS 10162-2 specifies the test must be stopped, it was only the data up to a displacement of 6 mm that were used for the analysis. As the force was applied to the specimen, the steel strips would slip from one thread to another as the screw tilted. This caused a lot of jumps in the force-displacement curve. The capacity was taken as the overall highest force reached before a displacement of 6 mm occurred. Figure 5.15 graphically illustrates some important points on a typical force-displacement curve used to determine the connection specimen capacity.

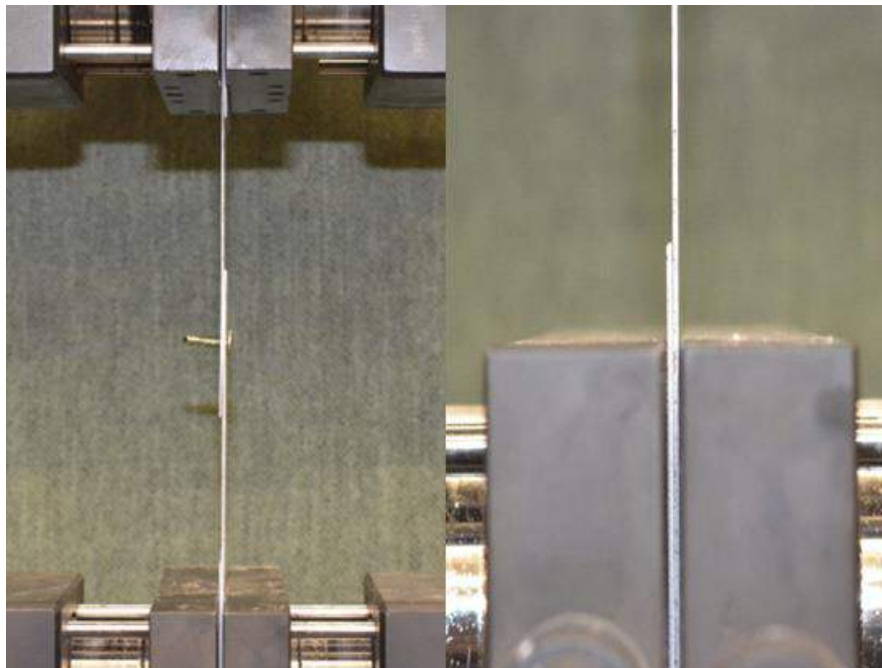


Figure 5.14 Original clamped in specimen (left) and end plate (right)

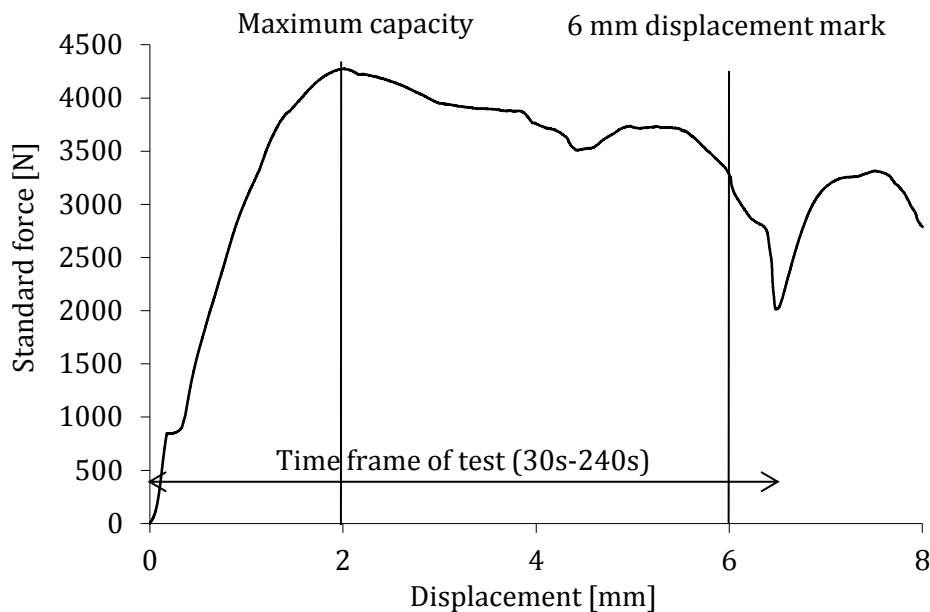


Figure 5.15 Typical force-displacement curve of tested connection specimen

5.4 Preliminary tests

Before the execution and during the planning of the final experiment, a number of questions and alternatives came forward. How much the steel strength varies within a coil, whether the steel coating had to be removed before or after performing tensile tests and what influence the screw thread grading has on the connection capacity are examples of such questions. In order to answer these questions, decide on which equipment and which procedure to follow, some trial tests were performed. In support of, and to justify the final experimental design, the primary preliminary tests are briefly explained in the following two sections.

5.4.1 Tensile testing

The steel thickness used in design and capacity predictions is the base metal thickness (BMT). In order to measure this thickness, the galvanised coating had to be removed. The first preliminary test was to determine whether the coating had to be removed prior to or after testing. Similar tests were done by Mr. Hendrik Christoffel Stephan (Stephan, 2013) who suggested that the coatings are removed after testing as he found that the thickness after testing would on average be the same as the original thickness. He also suggested that the acid used to remove the coating might have an influence on the material strength and therefore the coating must be removed after testing. After a number of tests it was clear that due to local elongation and necking the



thickness did not remain the same. No real difference was found between the tensile strengths from specimens treated with and not treated with the acid. This together with the fact that the coating does not contribute to the overall steel strength it was concluded that the coating would be removed and thickness measurements would be taken prior to testing.

Ideally it would have been the best to test the tensile strength of each of the individual steel strips of each of the connections. This however was not possible. Instead it was decided to test the steel properties at different locations within a coil to see how much it differs and at how many locations it would be necessary to test the strength to get a reasonable and accurate ultimate strength average value. After testing the tensile strength of a coil at four locations, situated 3 m apart, the average tensile strength was 624 MPa with a standard deviation of 4.5 MPa. The in-coil strength coefficient of variation (less than 1%) was found to be small enough to justify testing of the strength at only one or two locations.

5.4.2 Connection capacity testing

To ensure that the connection capacity tests resulted in accurate and reliable results the necessary specifications had to be met. This was made possible by performing some preliminary tests. One of the most important specifications was that the grips must be able to apply a uniform loading without slipping. Two clamping systems were available with the one being mechanically operated and the other hydraulically. Both of them delivered accurate results without allowing slippage to occur during a test. Due to the ease of erection and test set-up, the hydraulic clamping system was chosen.

The screws used for connection purposes in light steel framing are available in a wide range of sizes and thread grades. The thread is usually classified as either being fine or coarse. To keep the number of varying variables limited, it was decided to only use a single type of fastener for all tests. A number of specimens with 4.8 mm diameter screws, using both thread types were tested and resulted in similar maximum capacities. The only difference was the shape of the force-displacement curves where the coarser thread resulted in deeper and more widely spread (on the displacement axis) jumps in the measured force. The ultimate capacity was the only interest and with the thread grade not influencing this, it was decided to use the more generally available, finely threaded screws.

5.5 Experimental limitations

No real limitations were experienced regarding the test set-up and procedure of the experimental design as the design was fairly simple. The limitations experienced throughout the



experiment were mainly associated with the materials used. While the reliability of typical connections (i.e. those most used in practice) is of primary interest; to apply the results to the general use of CFS would require assessment across the range of what is allowed by SANS. Due to unavoidable imperfections, not all steel coils of a certain steel property set have the exact same thickness and strength and thus some variation in the actual properties exists. For the steel used in the tests to be representative of those in practice, this variation had to be accounted for.

To quantify this variation, multiple coils of each steel property set (Table 5.1) had to be used. This, however, was not possible for all steel property sets. In South Africa, the primary CFS used for light steel frame building is the 550 MPa grade steel with a 0.8 mm thickness. Due to the dominance of this steel, the other steel property sets were not as freely available. To have a more diverse set of results, CFS with different properties were tested, but due to little availability, the variation within a single steel property set could not be implemented by using multiple coils, but had to be done by using typical (published) statistical parameters, various statistical techniques and by making some assumptions. This is discussed in more detail in Chapter 7.

5.6 Experimental Results

The results of the tensile- and connection capacity tests described in Sections 5.2 and 5.3 are given in this section. Also part of the experimental work was the steel thickness and screw diameter measurements that are used in the capacity predictions (Section 6.2). The measurements of these geometric specimen properties are also given in this section. Due to a large amount of data, only typical results and summarised observations are presented here with the entire set of data given in Appendix A.

5.6.1 Connection tests

This section provides summarised results of five different typical connections (Table 5.2). These connections can be differentiated mainly based on their steel property set (Table 5.1) and connector type. A total of 237 connections were tested.

5.6.1.1 Tested connection capacities

Table 5.5 gives the average tested capacity for each of typical connections tested.



Table 5.5 Average tested capacities

Typical connection	Property set	Steel thickness (mm)	Steel grade (MPa)	Connector type	No. of specimens tested		Average tested capacity (N)	
1	1 (coil 1)	0.8	550	screw	15	132	3913	4086
1	1 (coil 2)	0.8	550	screw	87		4347	
1	1 (coil 3)	0.8	550	screw	15		4636	
1	1 (coil 4)	0.8	550	screw	15		4279	
2	2 (single coil)	0.58	550	screw	15		1872	
3	3 (single coil)	1.0	300	screw	30		4406	
4	4 (single coil)	0.8	230	screw	30		3023	
5	1 (coil 4)	0.8	550	Pop-rivet	15		3295	

5.6.1.2 Observed Failure modes

The failure mode observed for all 222 screwed specimens were a combination of screw tilting and hole bearing with the former being the dominant contributor. Figure 5.16 illustrates the observed failure mode.

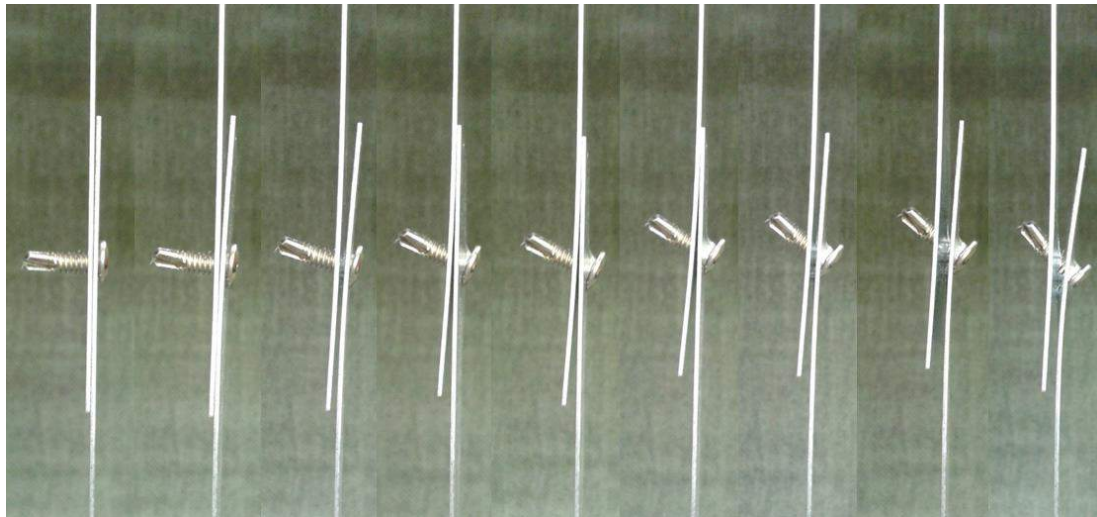


Figure 5.16 Observed failure mode (after every 1 mm displacement)

As the shear force was applied, the steel strips underwent some bearing and as the force increased, the screw started to tilt. Close up photos of the observed hole bearing, screw tilting and the combination of the two are given in Figure 5.18. As the screw tilted past a certain angle, it started to pull out. This pull out was caused by the force not being pure shear anymore, but rather a combination of shear and tension. It is important to note that the ultimate force was reached before any pulling out occurred. Using Figure 5.16, the maximum capacity was reached



at the fifth photo (5 mm displacement). The corresponding capacity curve is presented in Figure 5.17.



Figure 5.17 Typical capacity curve for connection test



Figure 5.18 Bearing (left), tilting (middle) and combination of tilting and bearing (right)

The 15 riveted connections failed in a somewhat similar manner as the screw connections, with tilting being the dominant failure mode. No real bearing was observed. As the force was applied, tilting started to occur and instead of pulling out (as the screws did), the pop rivet broke (failed in shear). The maximum capacity was reached during the tilting and had dropped off before the pop-rivet failed.

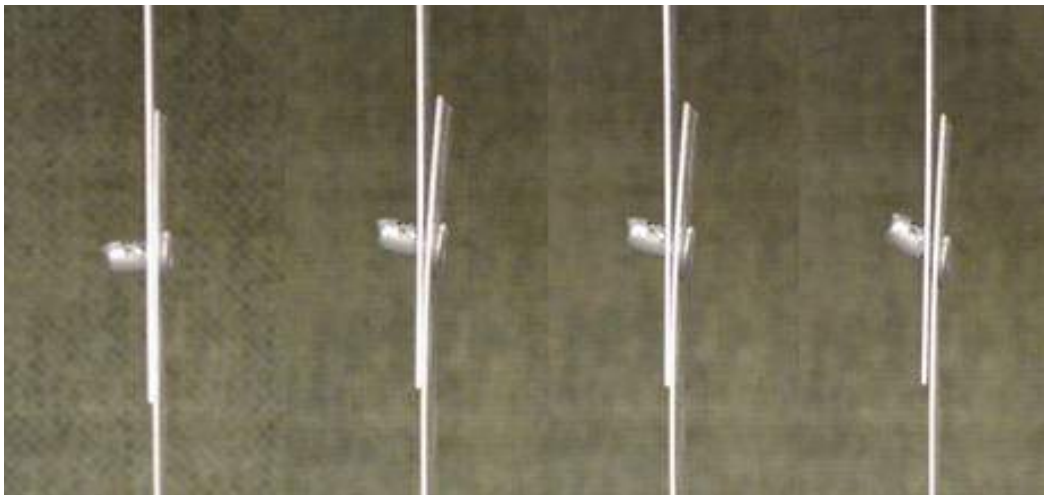


Figure 5.19 Riveted connection failure

5.6.2 Tensile tests

As discussed in Section 5.2, a tensile force was applied to a dog-bone shaped specimen until fracture occurred. Figure 5.20 shows an example of a fractured specimen. The specimen at the top is made from 300 MPa grade steel where the specimen at the bottom is made from 550 MPa grade steel. The influence of ductility is prominent in Figure 5.20 as the more ductile (300 MPa) steel clearly elongated more than the 550 MPa steel.

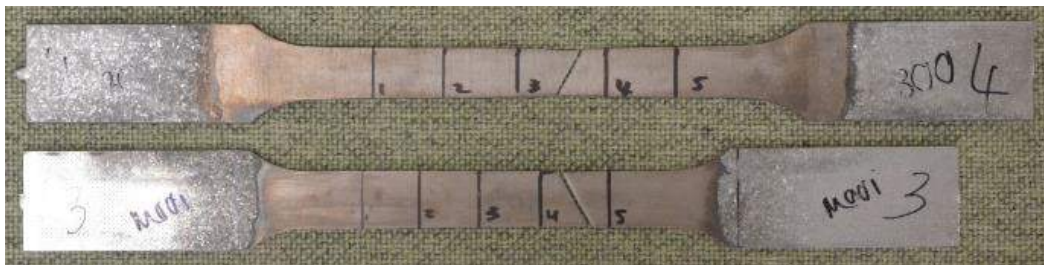


Figure 5.20 Fractured 300 MPa (top) and 550 MPa (bottom) specimens

5.6.2.1 Cross sectional area

The first step in determining the tensile stress for each of the steel grades was to determine the original cross sectional areas of the specimens. The average measured thicknesses and widths of the tensile test specimens are given in Table 5.6. The steel thickness and steel grade of the property sets in Table 5.6 are given in Table 5.5.

5.6.2.2 Ultimate tensile forces and stresses

The test results were obtained in the form of force-displacement curves. The force-displacement curves were converted to stress-strain curves with the use of the original cross section and the original length. Figure 5.21 show typical force-displacement curves for the steel property sets tested. As a specimen displacement (elongation) was measured with the crosshead separation,



the original length was taken as the initial distance between the gripped ends. Typical stress-strain curves are given in Figure 5.22 where the ultimate tensile force and stresses are summarised in Table 5.7. The force and stress (force/area) of each sample was calculated from which the average stresses were determined.

Table 5.6 Tensile test specimen original cross sections

Steel property set	No. of specimens tested	Average thickness (mm)	Average width (mm)	Average area (mm ²)
1 (coil 1)	6	0.769	20.02	15.39
1 (coil 2)	2	0.769	20.22	15.53
1 (coil 3)	2	0.754	20.07	15.10
1 (coil 4)	2	0.764	20.10	15.37
2 (single coil)	2	0.475	19.93	9.47
3 (single coil)	4	0.759	19.87	15.08
4 (single coil)	3	0.956	19.96	19.08

Table 5.7 Ultimate tensile forces and stresses

Steel property set	Average Ultimate tensile force (N)	Average ultimate tensile stress (MPa)	Ultimate tensile stress standard deviation (MPa)
1 (coil 1)	9325	606	3.28
1 (coil 2)	9832	633	2.87
1 (coil 3)	11501	762	5.82
1 (coil 4)	9681	630	1.42
2 (single coil)	5893	623	5.13
3 (single coil)	5977	396	1.01
4 (single coil)	7972	418	3.37

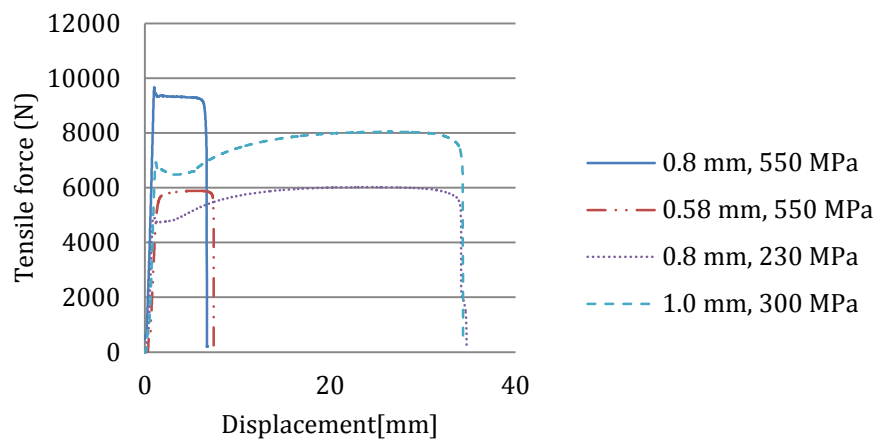


Figure 5.21 Typical force-displacement relationship

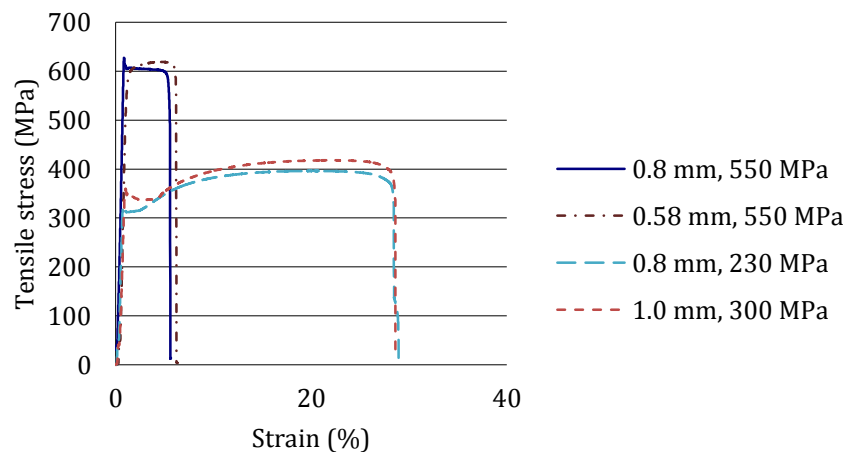


Figure 5.22 Typical stress-strain relationship

5.6.3 Thickness and diameter measurements

The measured nominal screw diameters of all the connection specimens were put into one data set of which an average and standard deviation was obtained. For the 222 screwed connections, an average screw diameter of 4.79 mm was obtained and an average diameter of 4.81 mm was measured for the 15 pop-rivets. The standard deviations for the screw and pop-rivet diameter measurements were 0.05 mm and 0.02 mm respectively. The average and standard deviation of the measured thicknesses of the connected members for the respective typical connections are given in Table 5.8. The properties of the different typical connections are given in Table 5.5.



Table 5.8 Connection specimen thickness measurements

Typical connection	Steel strip thickness (mm)			
	Average		Standard deviation	
1	0.76	0.77	0.003	0.007
1	0.77		0.002	
1	0.77		0.003	
1	0.76		0.004	
2	0.47		0.002	
3	0.76		0.005	
4	0.97		0.006	
5	0.76		0.003	

5.7 Concluding remarks

The tensile strength values obtained through tensile testing (Section 5.6.2), the average thickness values in Table 5.8 and the screw diameter measurements discussed in Section 5.6.3 are used in Chapter 6 as unbiased estimates of the input parameters in the capacity prediction model. These unbiased predictions together with the actual capacity test results (Section 5.6.1) are the used for the model factor determination.

Chapter 6

Determination of Model Factor

6.1 Introduction

In order to use a prediction model for reliable design, all unknown structural properties and uncertainties must be considered and quantified. This rational assessment of uncertainties is a fundamental part of a probabilistic design method. As discussed in Chapter 4, uncertainties directly associated with the prediction model, experimental design and test set-up are assessed using a model factor. To recap, a model factor is defined as the ratio between the predicted-and actual capacities, expressed as

$$\theta = \frac{V_{actual}}{V_{predicted}} \quad (6.1)$$

Using the actual capacity test results and the determined prediction model parameters presented in Chapter 5, this Chapter discusses the model factor determination of the SANS resistance prediction model for typical cold-formed steel (CFS) connections. For each of the typical connections tested in Chapter 5, the corresponding unbiased capacity prediction is first determined followed by the model factor calculations.

6.2 Capacity Predictions

In order for the prediction models to be declared reliable, it not only had to be able to predict the capacity, but also the failure mode. The predictions were done using the capacity formulas in SANS 10162-2. The predicted failure mode for the connections tested in this investigation, as discussed in Section 4.3, is a combination of screw tilting and hole bearing with tilting the main contributor. This failure mode is caused by the extreme thinness of the connected members that induces an eccentric shear load on the screw, causing it to tilt. For such a failure mode, the nominal capacity of screwed connections is predicted by

$$V_b = 4.2 \sqrt{t^3 d_f f_u} \quad (6.2)$$



where t is the thickness of the connected member not in contact with the screw head, d_f the nominal screw diameter and f_u the ultimate tensile strength of the connected members. When the nominal values of these parameters are replaced with unbiased estimates, Equation 6.1 may be used as an unbiased prediction model for connection capacities. For the current investigation, the measured steel thicknesses and screw diameters (Section 5.6) and the tensile strength obtained from tensile tests (Section 5.6) were used as the unbiased parameter estimates. The nominal capacity equation used for pop riveted connections is given as Equation 6.3 which is also representative of tilting and bearing. Once again, by using unbiased estimates of the input parameters, the nominal capacity equation delivers unbiased capacity predictions.

$$V_b = 3.6 \sqrt{t_2^3 d_f f_{u2}} \quad (6.3)$$

For each of the connections tested (Section 5.3), a corresponding capacity prediction was made using Equations 6.2 and 6.3. It is important to note that these predictions were made excluding the capacity reduction factor incorporated for design purposes. The average predicted capacities of the typical connections are given in Table 6.1.

Table 6.1 Average predicted capacities

Typical connection	Property set	Steel thickness (mm)	Steel grade (MPa)	Connector type	Average predicted capacity (N)	
1	1 (coil 1)	0.80	550	screw	3764	3892
1	1 (coil 2)	0.80	550	screw	3942	
1	1 (coil 3)	0.80	550	screw	4618	
1	1 (coil 4)	0.80	550	screw	3859	
2	2 (single coil)	0.58	550	screw	1854	
3	3 (single coil)	1.00	300	screw	3668	
4	4 (single coil)	0.80	230	screw	2412	
5	1 (coil 4)	0.80	550	Pop-rivet	3277	

6.3 Model factors

6.3.1 Results

A model factor for each of the 237 specimens was determined. These individual model factor values were interpreted as realisations of the overall model factor for each of the typical



connections (Table 6.1). This allowed for the model factor of each typical connection to be probabilistically expressed as a statistical distribution characterised by an average and standard deviation. An average model factor equal to one is an indication of a prediction model capable of producing unbiased predictions of the resistance capacity of a connection. When the model factor is less than one, it implies that the prediction model tends to yield capacities higher than the actual available capacities of connections i.e. unconservative. A model factor exceeding one implies a conservative tendency. According to Annex 4 of Reliability analysis for structural design, model factors typically average between 1 and 1.25 (Holicky, 2009). The calculated average model factors for each of the typical connections are given in Table 6.2. The model factor standard deviation indicated the prediction model's level of uncertainty. The standard deviations are given in Table 6.3.

Table 6.2 Calculated average model factors

Typical connection	Average predicted capacity (N)		Average tested capacity (N)		Average Model factor	
1	3764	3892	3913	4086	1.04	1.05
1	3942		4347		1.10	
1	4618		4636		1.00	
1	3859		4279		1.11	
2	1854		1872		1.01	
3	3668		4406		1.20	
4	2412		3023		1.25	
5	3277		3295		1.01	

6.3.2 Statistical properties

The main statistical properties used to characterise the model factor are the mean (μ) value of the model factor and the standard deviation (σ) or coefficient of variation ($w = \sigma/\mu$). The mean was determined by averaging all realisations of the model factor for each of the typical connection categories. The standard deviation and the coefficient of variation describe the dispersion of the data around the mean. Another way of describing the spread of the data is by using the minimum and maximum model factor values. The statistical characteristics of the model factors are given in Table 6.3.



Table 6.3 Model factors statistical properties

Typical connection	Mean μ		Standard deviation σ		Coefficient of Variation (%)		Min	Max
1	1.04	1.05	0.05	0.05	4.73	5.19	0.89	1.15
1	1.10		0.03		2.92			
1	1.00		0.04		4.24			
1	1.11		0.02		1.91			
2	1.01		0.06		6.03		0.92	1.19
3	1.20		0.05		4.38		1.10	1.31
4	1.25		0.08		6.03		1.09	1.36
5	1.01		0.05		5.19		0.90	1.08

6.3.3 Results discussion

The results given in Table 6.3 show that the resistance prediction model, irrespective of the steel properties, generally provide conservative predictions of connection capacities. Although some model factors were lower than one, all of the mean values exceeded one. The low standard deviations and coefficients of variance imply that the data is rather closely spread around the mean. This small spread of results is the result of a good prediction model with a low level of uncertainty as well as an accurate and constant testing procedure.

Some average model factors barely exceed unity and others have a conservative bias as high as 25 %, which implies that the level of conservatism varied as the steel properties differed. As the model factor values differed for different steel strength and thickness, possible trends of the model factor with the varying properties were investigated. Figure 6.1b shows the relationship between the ultimate tensile steel strengths and model factors with a clear trend of reduction in the model factor with increase in steel strength. A higher steel member thickness resulted in an increase in the model factor. This relationship is shown in Figure 6.1a. A trend in the model factor with any of the parameters (properties) indicates that the prediction model does not fully take account of the effect that the parameter has on the connection capacities.

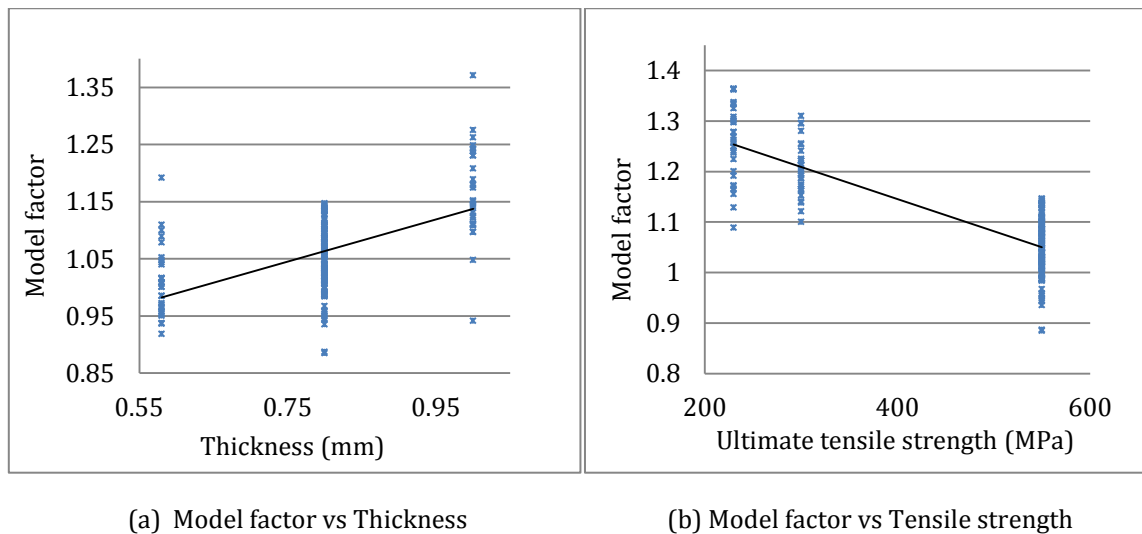


Figure 6.1 Model factor trends

Regarding screwed connections, the lowest model factor (1.01) was obtained when using the 0.58 mm steel. Comparing the conservative bias of the model factors for the different steel properties (screwed connections), the following statement could be made: Although the same resistance prediction model was used, the level of conservatism of the model depended on the steel properties. This statement can be supported by clause 1.5.1.4 of the SANS 10162-2. This clause states that when 550 MPa grade steel is used and the steel thickness is either less than 0.6 mm or 0.9 mm, the following modifications must be made.

- For 550 MPa grade steel with a thickness less than 0.9 mm, the steel strength used in the design calculations must be taken as the smallest of 495 MPa or 90% of the specified strength value.
- For 550 MPa grade steel with a thickness less than 0.6 mm, the steel strength used in the design calculations must be taken as the smallest of 410 MPa or 75% of the specified strength value.

By influencing the design capacity, these adjustments consequently influence the reliability. The reason why these adjustments are recommended is to take ductility into account. CFS loses its ductility, as the steel strength increases. This is the result of cold forming and other production processes. For the more ductile CFS, such as the 230 MPa and the 300 MPa steel, the above adjustments is not necessary as they still have an adequate amount of ductility. Getting back to how these adjustments are related to the level of conservatism of the model, these design modifications (reducing the design strength) can be seen as a way of making the connection more conservative. As it is only for these specific properties that the connection capacity is



made more conservative, it implies that the reduction in model conservatism for such properties is recognised.

6.3.4 Concluding remark

Unbiased predictions based on the SANS prediction model compared well with experimentally determined capacities, with average model factors (ratio of actual over predicted capacity) between 1.01 and 1.25 and standard deviations between 0.05 and 0.08. Model factors increased with increased plate thickness and decreased with increased steel strength. The level of conservatism of the prediction model depends on the material properties and was found to be at its lowest for steel with a strength of 550 MPa and thicknesses of 0.58 mm and 0.8mm. Very thin high strength plates have low ductility with little difference between the yield- and ultimate strength. Limitations of the nominal steel strength in design capacity computations for these ensure adequate conservatism, adjusting for the trend towards non-conservative unbiased predictions.

Chapter 7

Reliability Analysis

7.1 Introduction

Following a probabilistic approach in determining the reliability of structural resistance involves calculating the probability of failure and the corresponding reliability index by taking account of material-, geometric- and other uncertainties. Defined as the probability of the design resistance exceeding the actual resistance, the probability of failure is directly related to the reliability index, $p_f = \Phi(-\beta_t)$, where Φ is the standardised normal distribution. These reliability measures are seen as the most important parameters when it comes to structural reliability. In order for a structure or part of structure to be declared reliable, the determined reliability must meet the target reliability.

In South Africa the target reliability index for connections is $\beta_t = 4.5$, implying a target probability of failure of $p_f = 10^{-6}$. It is however routine to separate the reliability assessment of loads and resistance, by recognising that each of these components contribute in part to the total reliability. Their contributions are based on sensitivity factors which have been approximated and accepted (by the Eurocodes) as 0.8 and -0.7 for the resistance and loads respectively. Considering connections, the target resistance reliability is $\beta_{t,R} = 0.8 \times 4.5 = 3.6$ (Holicky, 2009; Holicky, 2013; SABS, 2011).

The determination of a reliability index and its corresponding probability of failure can be done through a structural reliability analysis. One of the most commonly used reliability analysis methods is the First Order Reliability Method (FORM). This method was used to determine the reliability index for typical screwed connections in cold formed steel (CFS) structures. The FORM analysis iteratively determines the reliability index for a given limit state function (LSF) until the smallest reliability index is found. All the necessary background information regarding the reliability theory was discussed in Chapter 3 which provides a good basis for the reliability analyses performed here.



7.2 General Limit State Function

Any structure or part of structure can be classified as either being reliable or, when not meeting the target reliability measures, as unreliable. Made up of different variables the LSF is a mathematical expression used to determine the reliability status of a structural part. For any part of structure, the probability of the design capacity exceeding the actual capacity must be acceptably small. Considering typical screwed connections, the limit state function G can be defined as the difference between the actual capacity of a connection and its design capacity.

$$G = V_{actual} - V_d \quad (7.1)$$

The actual capacity term should be representative of the true connection capacity taking all uncertainties into account. In this term, all material and geometric variables are represented in the form of statistical distributions. In addition, uncertainty not taken into account by the variable distributions was accounted for through the use of a model factor (Chapter 6), so that:

$$V_{actual} = \theta_{(\mu,\sigma)} * 4.2 \sqrt{t_{(\mu,\sigma)}^3 d_{f,(\mu,\sigma)} f_{u,(\mu,\sigma)}} \quad (7.2)$$

where θ is the model factor, t the thickness of the connected member not in contact with the screw head, d_f the nominal screw diameter and f_u the ultimate tensile strength of the connected members. Each of these variables was represented by statistical distributions and therefore required a set of data from which the necessary statistical parameters was determined. These data sets were obtained through physical measurement (Section 5.3), experimental work (Section 5.2) and literature. The terms in brackets in Equation 7.2 are indicating that the distributions of the variables were characterised by a mean μ and standard deviation σ . The design capacity V_d , on the other hand is determined as per SANS 10162-2 prescriptions, using characteristic/nominal values for the material and geometric properties and applying partial factors, reduction factors or material strength limits as required, so that:

$$V_d = \Phi * 4.2 \sqrt{t^3 d_f f_u} \quad (7.3)$$

The symbol Φ in Equation 7.3 is a capacity reduction factor applied to account for uncertainty in the resistance, including randomness, statistical limits, and model error (William et al, 2008). According to SANS 10162-2, the capacity reduction factor for screwed connections in shear is 0.5. This is discussed in Section 2.7. As a number of different steel property sets were used, the values of the design capacities and the distribution properties of the actual capacities were different, depending on the steel properties. Thus, with the basis and format of the LSF remaining constant throughout, a LSF for each of the CFS property sets was defined.



7.3 Material and geometric parameter uncertainties

A typical screwed connection consists of two or more members, connected with a single or multiple screw(s). For V_{actual} in Equation 7.1 to represent the best possible estimate of the true capacity, all uncertainties associated with the geometric and material parameters had to be rationally assessed which was done by modelling each of the parameters with an appropriate statistical distribution. Before discussing such a rational assessment, it is worth reviewing the different steel properties considered for the current investigation. The different steel property sets considered for this study are similar to those used in the South African engineering practice. The properties of the steel and also the number of steel coils tested of each property set are given in Table 7.1.

Table 7.1 Steel properties

Steel property set	Steel properties (coated thickness, steel grade)	No. of steel coils
1	0.8 mm, 550 MPa	4
2	0.58 mm, 550 MPa	1
3	1.0 mm, 300 MPa	1
4	0.8 mm, 230 MPa	1

For the steel considered here to be truly representative of those in practice, the variation in the actual steel properties of coils had to be accounted for. This variation includes both the variation within a single coil (in-coil variability) and the variation among numerous coils (between-coil variability) with the same nominal properties. To adequately quantify the total variability, multiple coils of each steel property set had to be used. This, however, was not possible for all property sets. In South Africa, the primary CFS used for light steel frame building is the 550 MPa grade steel with a 0.8 mm thickness. Due to the dominance of this steel, the other steel property sets were not as freely available. To add to the poor availability, the local suppliers make use of large coil sizes which led to a slow coil turnaround. Thus, excluding the 0.8 mm-550 MPa steel, the variation within a single steel property set, could not be implemented by using multiple coils, but had to be done by using generally used statistical parameters, through the application of various statistical techniques and by making appropriate assumptions.

When measurements from a single coil are considered it is clear that the between-coil variability is under estimated and will result in low estimates of the actual steel properties standard deviation. When using measurements from four coils, some of the between-coil



variability is captured, but the quality of the estimate may be subject to sample error (too small a sample to get good standard deviation estimate). The statistical parameters derived from the four-coil measurements were compared to general statistical parameters found in literature in order to determine whether the four coils represent an adequate amount of variability in the material properties.

7.3.1 Geometric parameters

The quantification and assessment of the uncertainties associated with the thickness and screw diameter was done through probabilistic modelling. The chosen distribution, as shown in Equation 7.2, was characterised by an average and standard deviation which was obtained from a set of data. The data sets used for determining these statistical parameters of the thickness and diameter distributions were obtained through physical measurement, as discussed in Section 5.3.

7.3.1.1 Member thickness

The actual thickness of a member is typically slightly larger than the claimed nominal thickness. According to various literatures, the variability in structural dimensions tends to be small and can adequately be modelled by a normal distribution. Modelled as a normal distribution, the coefficient of variation (CoV) for steel geometry usually ranges between 1% and 4% (Holicky, 2009; JCSS, 2000). These suggested CoV values were used to compare the measured values and motivate their use in the analyses. In order to compare the CoV values used for the analyses to this proposed range (1-4%) and for the tested steel to simulate those in practice, the calculated steel CoV (average and standard deviation) values had to include an appropriate level of both the in-coil variability and the between-coil variability. Referring to Table 7.1, the inclusion of this variability was rather easily done for the steel category (0.8 mm, 550 MPa) with four coils, but a bit more complex for the property sets of which only a single coil was tested.

A total of 132 connection test specimens (Chapter 5) had 0.8 mm thick members with a steel grade of 550 MPa. Thus, including all four coils, a total of 132 thickness measurements were obtained for the 0.8 mm, 550 MPa steel. The measured thicknesses of this steel property set were, at first, kept separate according to the respective coils. A total of 87 measurements were obtained from one coil and 15 measurements from each of the other three coils. An average thickness for each of the four coils was obtained. These four average thicknesses were seen as four sample points. The average μ and standard deviation σ of these four sample points was seen as the overall thickness statistical parameters and were used to characterise the thickness distribution for the 0.8 mm, 550 MPa steel. Table 7.2 gives the single coil thickness averages



together with the combined average μ and standard deviation σ that were used as the distribution characteristics in the reliability analysis. Due to the manner in which these distribution characteristics were obtained, it can no longer be claimed that the thickness in-coil variability is taken into account by the thickness distribution. By using the average thickness of each coil as a sample point which in turn was used to determine the distribution parameters, the in-coil variability is not accounted for by the distribution. As the individual specimen measurements were used for the capacity predictions (Section 6.2) and not the average thickness per coil it implies that the in-coil thickness variability is also excluded from the model factor. Thus, the in-coil variability of the thickness was not taken into account during analysis, but with the thickness variability within a single coil being extremely low ($\text{CoV} < 0.85\%$) it was safe to assume that the neglecting of this variability would not influence the investigated reliability.

Table 7.2 Thickness statistical parameters for 0.8 mm, 550 MPa steel

Coil no.	Single coil μ (mm) (sample point)	Combined μ (mm)	Combined σ (mm)	Combined CoV (σ/μ) (%)
1	0.771	0.765	0.007	1
2	0.771			
3	0.757			
4	0.762			

Although the value of 1% falls within the range suggested by literature, the sample size is too small to obtain a standard deviation, with great confidence, representative of the entire population (steel in general). For the four coil sample a confidence interval on the standard deviation of the sample thickness ranging from 0.005 to 0.15 is obtained when using a confidence level of 75%. In other words, it can thus be said that the thickness standard deviation of 0.8 mm, 550 MPa CFS in general lies in between 0.005 mm and 0.015 mm with 75% confidence.

When using a single coil, the level of variability is too small and does not account for uncertainties caused by actual property differences among coils that are nominally the same. An adequate level of variability had to be implemented by means of statistics and assumptions, due to the lack of appropriate data. The statistical parameters necessary to characterise the thickness distributions of steel property sets 2-4 was obtained by using the average and standard deviation of the 0.8 mm, 500 MPa steel as basis.



When the actual thickness of a steel coil is measured, it will more often than not deviate from the nominal value. This deviation is primarily caused during production by the tolerances/accuracy of the plate-rolling machines. The same machines are used for all thicknesses which allows for the assumption to be made that the effect of the machine inaccuracies on the steel thickness is the same for all thicknesses and thus, the standard deviation should have similar absolute values irrespective of the nominal thickness of the plate. This assumption was used as motivation behind the procedure followed in determining the thickness average and standard deviation for the three single coil steel property sets.

The above assumption regarding the standard deviation allowed for the standard deviation obtained from the 0.8 mm, 550 MPa steel to be used for the remaining three steel property sets. To include the variability in thickness among different coils, the combined standard deviation of the 0.8 mm, 550 MPa steel were used. Table 7.3 shows the statistical parameters used to characterise the thickness distributions for all the tested steel property sets. Also given in Table 7.3 are the thickness distribution averages. The determination of these averages for steel property sets 2-4 is discussed below.

As it was assumed that the margin by which the actual measured thickness deviates from the nominal thickness is the same for all thicknesses, it can also be said that the relationship (ratio) of the actual measured thickness and the nominal thickness would be the same for all thicknesses. Thus, when this relationship is known for any of the steel property sets, it can be applied to the remaining sets to obtain the average actual thicknesses with all the nominal thicknesses known (provided by steel suppliers). This relationship for the 0.8 mm, 550 MPa steel can be calculated as both the nominal and actual thickness values are available. The nominal thicknesses are provided by the steel suppliers and the actual thickness (average measured value) of the 0.8 mm, 550 MPa was obtained as discussed earlier in this section (Table 7.2). The nominal thicknesses used here are the base metal thickness (BMT) which is the total coated thickness minus the prescribed coating thickness of 0.04 (SANS 517). Quantified as a bias estimator (on the nominal values), the relationship for the 0.8 mm, 550 MPa steel was determined as,

$$\text{Bias} = \frac{\text{Measured average of actual BMT}}{\text{Nominal value of BMT}} = \frac{0.765}{0.760} = 1.01$$

By multiplying this bias to the nominal BMT of the each of the property sets, an average thickness value for each of the sets was obtained. These average values together with the standard deviations were used to characterise the statistical distributions. The average



thicknesses of the single coils obtained from measurements (Chapter 5) were not used as the distribution averages as it was argued that they do not account for the between-coil variability. These averages, was however, used to compare the estimated average values for validation (Table 7.3). The Nominal BMT values in Table 7.3 are the total coated thicknesses, (obtained from supplier) minus the prescribed coating thickness of 0.04 (SANS 517).

Table 7.3 Thickness distribution parameters

Steel property set (coated thickness, steel grade)	Nominal BMT (mm)	Bias	Distribution parameters used in analyses		Measured μ (mm) (single coil)
			μ (mm) (Nominal BMT \times Bias)	σ (mm)	
1 (0.8 mm, 550 MPa)	0.76	1.01	0.765	0.007	0.765
2 (0.58 mm, 550 MPa)	0.54	1.01	0.55	0.007	0.47
3 (1.0 mm, 300 MPa)	0.96	1.01	0.97	0.007	0.97
4 (0.8 mm, 230 MPa)	0.76	1.01	0.765	0.007	0.76

7.3.1.2 Screw diameter

The diameter of a screw is also seen as a structural dimension and thus, as for the thickness, the screw diameter was modelled using a normal distribution. Using the range of typical CoV values discussed under 7.3.1.1, the adequacy of the measured CoV was checked.

A total of 300 screws were measured. The average and standard deviation of these 300 individual samples were used to characterise the diameter random variable. The screw diameter average and standard deviation are 4.79 mm and 0.05 mm respectively. The resulting CoV of 1% did fall within the proposed range of 1-4% (Holicky, 2009). The same was done here that was done for the thickness regarding the confidence interval on the measured sample standard deviation. The manner in which the above average and standard deviation was calculated implies a sample size of 300 which led to a smaller confidence interval. The population standard deviation (screws in general) lies in between 0.048 mm and 0.052 mm with a 75 % confidence.

The calculation above of the standard deviation of the screw diameter probably underestimated the appropriate variability found in practice. The screws were obtained from the same supplier and were therefore likely made by the same machine, but from different base metals. Some correlation and dependence are expected between the diameter measurements and thus it is incorrect to interpret all measurements as individual independent samples. This implies that the standard deviation was underestimated which led to questioning and investigating the



influence of this underestimation on the reliability results as the analyses were performed using the initial calculated average and standard deviation. The analysis of typical connection 1 (Section 7.5.3) was redone using the same average as above, but a larger CoV and thus also a larger standard deviation (0.1916 mm). The upper limit of the range proposed by Holicky was used in order to see whether the higher variability had a considerable influence on the reliability. The CoV of 4% resulted in a reliability index of 8.23 which is insignificantly lower than the original 8.33 obtained for typical connection 1, thus it was decided to keep all results as it was initially calculated.

7.3.2 Steel strength

This section handles the determination of the average and standard deviation that were used to characterise the ultimate tensile strength statistical distribution for each of the steel property sets given in Table 7.1. According to various literature, a log-normal distribution is used for modelling the strength of steel and usually has a CoV of 7-10% (Holicky, 2009; JCSS, 2000).

If a large amount of coils were available, the average and standard deviation of the tensile strength could easily be determined. By testing multiple coils, the appropriate amount of variability between coils is automatically implemented. This, however, was not the case. The amount of coils, and thus also measurements, was restricted to only a single coil for each of the steel property sets shown in Table 7.1 except for steel with a thickness of 0.8 mm and a steel grade of 550 MPa. This steel is the most frequently used steel in practice, and was thus more freely available. For the steel strengths of which only one coil were used, an adequate level of variability had to be implemented by means of statistical inference and assumptions due to this lack of appropriate data.

Regarding the four coils of 0.8 mm thickness and 550 MPa steel grade, it could be argued that the data set (sample size of four) was still too small for the results to be used as representative of the materials used in practice. One problem of using a too small sample size to represent a population is an incorrect variation. To check whether the average and standard deviation obtained from the tests (of the four coils) can adequately be used to represent the steel in general it was compared to values generally used to characterise tensile strength distributions. Due to the small sample size (four), it is unlikely to obtain a sample standard deviation, truly representative of the population standard deviation, with great confidence. Therefore a confidence interval was determined as an indication of the certainty of the calculated standard deviation. Using the sample (four coils) standard deviation calculated below and referring back to the previous section where this was done for the thickness and screw diameter, the



population (steel in general) standard deviation of the ultimate tensile strength can be said to be between 51 MPa and 148 MPa with 75% confidence. The following paragraph discusses the procedure followed in obtaining the CoV (average and standard deviation) from the four 0.8 mm, 550 MPa steel coils.

After a number of preliminary tests (Section 5.4) where the steel strength was tested at various locations within a coil, a small coefficient of variation (CoV) of less than 1% was observed. Thus, the in-coil variability was found to be small. It was therefore decided to only test the steel strength at three locations of which an averaged value was determined and used as the average strength of that particular coil. Regarding the 0.8 mm, 500 MPa steel, the four average values obtained from the four coils were seen as four sample points. The average and standard deviation of these four points were used as the average and standard deviation of the tensile strength statistical distribution for the 0.8 mm, 550 MPa steel. These distribution properties are given in Table 7.4.

As the average calculated tensile strength of each coil was used in determining the tensile strength distribution parameters, the in-coil uncertainty of the tensile strength could not be included in the tensile strength distribution, but rather was accounted for by the model factor. To recap, the model factor is representative of uncertainties that causes the predicted capacity to deviate from the actual capacity and that are excluded from the variable (thickness, diameter, strength) distributions. As the average tensile strength of each coil was used for the connection capacity predictions, it does not represent the exact strength of the specimen and thus contributes to the predicted capacity deviating from the true capacity. Therefore, the in-coil variability of the tensile strength was taken into account by the model factor.

Although it is only the ultimate tensile strength discussed here, the same procedure was followed in determining the yield strength which was necessary for determining and validating the distribution average and standard deviation of the three remaining steel property sets. The yield strength average and standard deviation of the four coils were 656.1 MPa and 71 MPa respectively.

As the CoV of 10% falls on the conservative side of the range suggested by literature, the average and standard deviation in Table 7.4 were used to represent the ultimate tensile strength of 0.8 mm, 550 MPa steel. Due to the fact that this steel had four coils, the procedure in determining the statistical parameters of the distribution was fairly simple as it automatically included the variability and uncertainty among coils of the same nominal properties. For the



three steel property sets of which only one coil was tested, the procedure needed additional statistical techniques and assumptions.

Table 7.4 Ultimate tensile strength statistical parameters for 0.8 mm, 550 MPa steel

Coil no.	Single coil μ (MPa) (sample point)	Combined μ (MPa)	Combined σ (MPa)	Combined CoV (%)
1	606	657.7	70.5	10
2	633			
3	762			
4	630			

The biggest problem of only having a single coil is that there is only a single sample point and thus, no standard deviation can be determined that would include the tensile strength variability and uncertainty among multiple coils of the same nominal properties. This was important as the variable distributions needed to be representative of the entire population (steel in general) and not just a small sample. The solution was thus to find values for both the average and standard deviation of the tensile strength that would represent a wide range of steel and include an appropriate level of variability in the tensile strength.

The assumption regarding the standard deviation of the thicknesses could not be made here as the difference between the actual steel strength and the nominal strength depend on more than just the accuracy of the plate-rolling machines. Therefore, an alternative way of including the between-coil variability had to be made. This was done using Equations 7.4 and 7.5 below, from Annex 4 of the *Reliability analysis for structural design* (Holicky, 2009). In this Annex the characteristic strength refers to the nominal values used in design calculations and unlike the generally used 5% fractile, the characteristic strength for steel in this annex is taken as the 1% fractile of a log-normal distribution. The ultimate tensile strength and yield strength are generally described by log-normal distributions (Holicky, 2009).

$$f_{y,\mu} = f_{y,k} + C1 \times \sigma_y \quad (7.4)$$

$$f_{ult,\mu} = f_{ult,k} + C2 \times \sigma_{ult} \quad (7.5)$$

$$f_{ult,\mu} = \kappa \times f_{y,\mu} \quad (7.6)$$

In these equations, the subscripts y and ult stand for yield and ultimate where μ and k stand for mean and characteristic (nominal). $C1$ and $C2$ are constants that represent the number of standard deviations between the average and characteristics values. As it is only the ultimate tensile strength used in the analyses, the yield strength was only determined to support and



motivate the determined ultimate tensile strength parameters. The symbol κ (kappa) in Equation 7.6 is used to quantify the relationship between the yield and tensile strength. Equations 7.4 and 7.5 represent the relationship between the characteristic and mean values of a distribution. This is graphically illustrated in Figure 7.1 where two log-normal distributions are plotted.

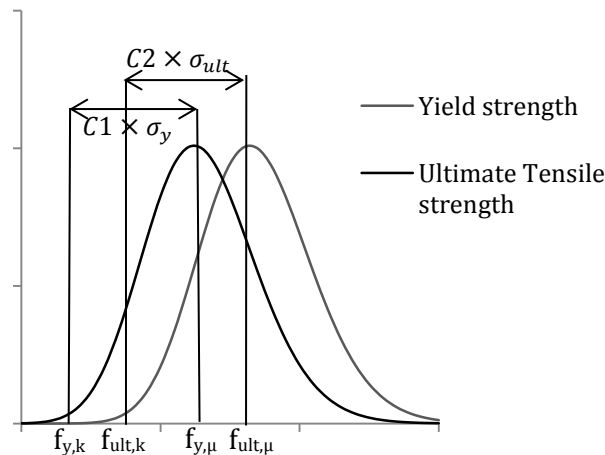


Figure 7.1 Relationship between mean and characteristic values

Table 7.5 Characteristic strength values

Steel property set (coated thickness, steel grade)	$f_{y,k}$ (MPa)	$f_{ult,k}$ (MPa)	κ ($f_{ult,k}/f_{y,k}$)
1 (0.8 mm, 550 MPa)	550	550	1.00
2 (0.58 mm, 550 MPa)	550	550	1.00
3 (1.0 mm, 300 MPa)	300	340	1.13
4 (0.8 mm, 230 MPa)	230	310	1.35

The characteristic values ($f_{ult,k}, f_{y,k}$) is known from the SANS 10162-2 and is also provided by steel suppliers. These are given in Table 7.5. Regarding steel property sets 2-4 (Table 7.1), the average (f_{μ}) and standard deviation (σ) of both the ultimate and yield strength is unknown and needs to be determined from measurements and assumptions. The values of $C1$ and $C2$ is also unknown for these property sets, but can be obtained for the 0.8 mm, 550 MPa steel as both the characteristic (Table 7.5) and average values (Table 7.4) of this steel is known. For the yield strength (Equation 7.4):

$$656.1 = 550 + C1 \times 71$$

$$C1 = 1.5$$



For the ultimate tensile strength (Equation 7.5):

$$657.7 = 550 + C2 \times 70.5$$

$$C2 = 1.5$$

When using a log-normal distribution, the fractile corresponding to 1.5 standard deviations is the 5% fractile for a skewness of more or less 0.5. Although some literature (Holicky, 2009) uses the 1% fractile, the characteristic value of material properties is typically assumed as the 5% fractile. It can thus be said that the measurements on the four coils of 0.8 mm, 550 MPa steel confirm that for the CFS coils used, the 5% fractile assumption is valid. By assuming that this fractile stays the same irrespective of the CFS properties, the number of standard deviations ($C1$ and $C2$) equivalent to the 5% fractile then also stays the same (equal to 1.5) for all property sets. Thus, the values of $C1$ and $C2$ are equal to 1.5 for all steel property sets.

As the fractile represents the relationship between the average-and characteristic strength values, it can be said that the ratio (another representation of the relationship) of the two values will also remain the same for all steel property sets. By using Tables 7.4 and 7.5, this ratio can be determined for the 0.8 mm, 550 MPa steel and can then be applied to the remaining sets to obtain the average actual strength values with all the nominal (characteristic) strength values known (Table 7.5). Quantified as bias estimators (on the nominal values), the ratios of the 0.8 mm, 550 MPa steel was determined as

$$\text{Bias of yield strength} = \frac{\text{Measured average}}{\text{Characteristic value}} = \frac{656.1}{550} = 1.19$$

$$\text{Bias of ultimate tensile strength} = \frac{\text{Measured average}}{\text{Characteristic value}} = \frac{657.7}{550} = 1.19$$

By multiplying the characteristic values with the bias, the average values of each of the remaining steel property sets was obtained. It is worth mentioning that these computed averages compares well with the average measured strength obtained for each of the single coils. The single coil measured averages are included in Tables 7.6 and 7.7 for comparison with the values computed using the bias and the nominal strength values. The reason why the single coil averages were not used is because it does not account for between-coil variability. The characteristic values, bias and the average values are shown in Tables 7.6 and 7.7. Also included in these tables are the standard deviations that were calculated by solving Equations 7.4 and 7.5 with the use of the averages (Tables 7.6 and 7.7), constants ($C1$ and $C2$) and characteristic values (Table 7.5). These values include an appropriate level of variability which allows them to represent a wide range of coils.



Table 7.6 Ultimate tensile strength distribution parameters

Steel property set (coated thickness, steel grade)	Characteristic strength $f_{ult,k}$ (MPa)	Bias	Distribution parameters used in analyses		Single coil measured average (MPa)
			Average strength $f_{ult,\mu}$ (MPa) ($f_{ult,k} \times \text{Bias}$)	σ_{ult} (MPa)	
1 (0.8 mm, 550 MPa)	550	1.19	658	71	658
2 (0.58 mm, 550 MPa)	550	1.19	658	71	623
3 (1.0 mm, 300 MPa)	340	1.19	408	45	418
4 (0.8 mm, 230 MPa)	310	1.19	372	41	396

Table 7.7 Yield strength distribution parameters

Steel property set (coated thickness, steel grade)	Characteristic strength $f_{y,k}$ (MPa)	Bias	Distribution parameters used in analyses		Single coil measured average (MPa)
			Average strength $f_{y,\mu}$ (MPa) ($f_{y,k} \times \text{Bias}$)	σ_y (MPa)	
1 (0.8 mm, 550 MPa)	550	1.19	656	71	656
2 (0.58 mm, 550 MPa)	550	1.19	656	71	578
3 (1.0 mm, 300 MPa)	300	1.19	357	29	339
4 (0.8 mm, 230 MPa)	230	1.19	274	38	311

As it is only the ultimate tensile strength necessary for the reliability analyses, the yield strength was calculated with the purpose of checking whether the calculations delivered satisfactory results using Equation 7.6. The kappa (κ) parameter in Equation 7.6 gives the relationship between the ultimate and yield strength. Equation 7.6 was used to check whether the kappa values resulting from the computed average values from Equations 7.4 and 7.5 matches the characteristic kappa values from literature (SANS 10162-2). The kappa values of the calculated average strengths was the same as the characteristic values kappa values which is confirmation that the average calculations resulted in an satisfactory relationship between the yield and ultimate tensile strength.



Table 7.8 Kappa values

Steel property set (coated thickness, steel grade)	Characteristic values obtained from literature			Average values calculated		
	$f_{y,k}$ (MPa)	$f_{ult,k}$ (MPa)	κ (k)	$f_{y,\mu}$ (MPa)	$f_{ult,\mu}$ (MPa)	κ (μ)
1 (0.8 mm, 550 MPa)	550	550	1.00	656	658	1.00
2 (0.58 mm, 550 MPa)	550	550	1.00	656	658	1.00
3 (1.0 mm, 300 MPa)	230	310	1.35	357	408	1.35
4 (0.8 mm, 230 MPa)	300	340	1.13	274	372	1.13

7.4 Model factor

Determined as the ratio of the tested capacity over the corresponding predicted capacity, a model factor was determined for 222 single-point screwed connections and 15 riveted connections. The model factor was included in the actual capacity term (Equation 7.2) for the assessment of the uncertainties directly associated with the prediction model, uncertainties due to experimental variability and also the uncertainties caused by the strength variability within coils. The model factor determination and discussion thereof is given in more detail in Chapter 6 with only a summary of the statistical parameters given here. The calculated model factor distribution parameters for the typical CFS connections are shown in Table 7.8. According to Annex 4 of *Reliability analysis for structural design* (Holicky, 2009), model factors average between 1 and 1.25 and is adequately modelled by a normal distribution with a CoV of 4-16%.

Table 7.9 Model factor distribution parameters

Typical connection	Property set	Steel thickness (mm)	Steel grade (MPa)	Connector type	Model factor		
					μ	σ	CoV (%)
1	1	0.80	550	screw	1.05	0.055	5.19
2	2	0.58	550	screw	1.01	0.060	6.03
3	3	1.00	300	screw	1.20	0.053	4.38
4	4	0.80	230	screw	1.25	0.076	6.03
5	1	0.80	550	Pop-rivet	1.01	0.050	5.19



7.5 Reliability analyses

7.5.1 Screwed connections

In order for typical screwed connections in tilt-and bearing, designed according to SANS, to be considered sufficiently reliable, the computed resistance reliability index β_R must exceed the specified target reliability index $\beta_{t,R} = 3.6$ for connections in South Africa. Chapter 3 discussed the literature behind a typical reliability analysis and serves as a good basis for the analyses performed here. This section provides the summarised input data, the reliability results and a results discussion.

The limit state function (LSF) is the basis of any reliability analysis and for the current analysis the LSF is defined as the difference between the actual connection capacity and the design capacity. The design capacity V_d is computed deterministically based on the design provisions of SANS 10160-2 and the actual capacity was probabilistically modelled using appropriate statistical distributions. Equations 7.7 and 7.8 give the general expression of the LSF G .

$$G = V_{actual} - V_d \quad (7.7)$$

$$G(\text{screwed connection}) = \theta_{(\mu,\sigma)} \times 4.2 \sqrt{t_{(\mu,\sigma)}^3 d_{f,(\mu,\sigma)} f_{u,(\mu,\sigma)}} - \Phi \times 4.2 \sqrt{t^3 d_f f_u} \quad (7.8)$$

The distribution characteristics of the input parameters for all the tested steel categories are given in Table 7.10 where the corresponding design values are given in Table 7.11.

Table 7.10 Actual capacity distribution properties

Steel property set (Thickness, steel grade)	Thickness (mm)		Screw Diameter (mm)		Tensile strength (MPa)		Model factor	
	μ	σ	μ	σ	μ	σ	μ	σ
1 (0.8 mm, 550 MPa)	0.765	0.007	4.79	0.05	658	71	1.05	0.055
2 (0.58 mm, 550 MPa)	0.55	0.007	4.79	0.05	658	71	1.01	0.060
3 (1.0 mm, 300 MPa)	0.97	0.007	4.79	0.05	408	45	1.2	0.053
4 (0.8 mm, 230 MPa)	0.77	0.007	4.79	0.05	372	41	1.25	0.076
Distribution	Normal		Normal		Log-Normal		Normal	



Table 7.11 Design values and capacities

Steel property set (Thickness, steel grade)	Design values				
	ϕ	d_f (mm)	t (mm)	F_u (MPa)	Capacity (N)
1 (0.8 mm, 550 MPa)	0.5	4.8	0.76	495	1508.92
2 (0.58 mm, 550 MPa)	0.5	4.8	0.54	410	748.54
3 (1.0 mm, 300 MPa)	0.5	4.8	0.96	340	1471.38
4 (0.8 mm, 230 MPa)	0.5	4.8	0.76	310	944.98

7.5.2 Riveted connections

In addition to the screwed connections, a total of 15 riveted connections were also tested and analysed. The distributions and their properties used in the LSF for riveted connections are given in Table 7.12. Steel of 0.8 mm thickness and grade of 550 MPa were used for the riveted connections.

Table 7.12 Distribution properties for riveted connections

Variable	μ	σ	Distribution
θ	1.01	0.05	Normal
t (mm)	0.77	0.007	Normal
d_f (mm)	4.81	0.02	Normal
f_u (MPa)	658	71	Log-normal

The design capacity of riveted connections for CFS structural members was calculated using Equation 7.9. The LSF used for the reliability analysis is given as Equation 7.10.

$$V_d = \Phi 3.6 \sqrt{t^3 d_f f_{u2}} = 0.5 \times 3.6 \sqrt{0.76^3 \times 4.8 \times 495} = 1293 \text{ N} \quad (7.9)$$

$$G(\text{riveted}) = \theta_{N(1.01,0.05)} \times 3.6 \sqrt{t_{N(0.77,0.007)}^3 d_{f,N(4.81,0.02)} f_{u,LN(658,71)}} - 1293 \quad (7.10)$$

7.5.3 Analyses Results

The reliability analysis was done using a computer software package called VaP (the Variable Processor). The following results were obtained:

- Reliability index and associated probability of failure
- FORM sensitivity factors
- Design point values



Tables 7.13-7.18 show the results from the reliability analyses for the typical connections.

Table 7.13 Typical connection 1 (Screwed, 0.8mm, 550MPa)

Reliability Index (β): 8.33		Failure probability: $4.2e^{-17}$
Variable	Design value	Sensitivity factor
Tensile strength (MPa)	308.56	-0.84
Model factor	0.81	-0.53
Screw diameter (mm)	4.77	-0.04
Base metal thickness (mm)	0.76	-0.11

Table 7.14 Typical connection 2 (Screwed, 0.58mm, 550MPa)

Reliability Index (β): 9.15		Failure probability: $2.96e^{-20}$
Variable	Design value	Sensitivity factor
Tensile strength (MPa)	310.98	-0.76
Model factor	0.66	-0.64
Screw diameter (mm)	4.77	-0.04
Base metal thickness (mm)	0.54	-0.14

Table 7.15 Typical connection 3 (Screwed, 1.0mm, 300MPa)

Reliability Index (β): 8.84		Failure probability: $2.6e^{-17}$
Variable	Design value	Sensitivity factor
Tensile strength (MPa)	169.77	-0.9
Model factor	0.997	-0.43
Screw diameter (mm)	4.77	-0.04
Base metal thickness (mm)	0.97	-0.09

Table 7.16 Typical connection 4 (Screwed, 0.8mm, 230MPa)

Reliability Index (β): 8.4		Failure probability: $2.27e^{-17}$
Variable	Design value	Sensitivity factor
Tensile strength (MPa)	181.50	-0.77
Model factor	0.85	-0.63
Screw diameter (mm)	4.77	-0.04
Base metal thickness (mm)	0.76	-0.1



Table 7.17 Typical connection 5 (Riveted, 0.8mm, 550MPa)

Reliability Index (β): 8.25		Failure probability: $8.24e^{-17}$
Variable	Design value	Sensitivity factor
Tensile strength (MPa)	304.86	-0.86
Model factor	0.81	-0.497
Rivet diameter (mm)	4.81	-0.017
Base metal thickness (mm)	0.764	-0.11

Table 7.18 Summarised analysis results

Typical connection	Tested resistance reliability index β_R	Target resistance reliability index $\beta_{t,R}$	Comparison
1	8.33	3.6	$\beta_{t,R} \gg \beta_R$
2	9.15	3.6	$\beta_{t,R} \gg \beta_R$
3	8.4	3.6	$\beta_{t,R} \gg \beta_R$
4	8.84	3.6	$\beta_{t,R} \gg \beta_R$
5	8.25	3.6	$\beta_{t,R} \gg \beta_R$

Table 7.18 shows that all computed reliability indexes exceed the target index by a significant margin. During design, a capacity reduction factor of 0.5 is applied to account for uncertainty in the resistance, including randomness, statistical limits, and model error (William et al, 2008). The application of this factor was identified as one of the primary causes for the actual capacity far exceeding the design capacity. The influence of this factor is made clear in Figure 7.2 where the tested capacities are plotted together with the nominal and design capacities. The range denoted as (a) represents the conservative bias achieved through the use of nominal values and other conservative limitations where (b) is an additional conservative bias achieved by applying the capacity reduction factor.

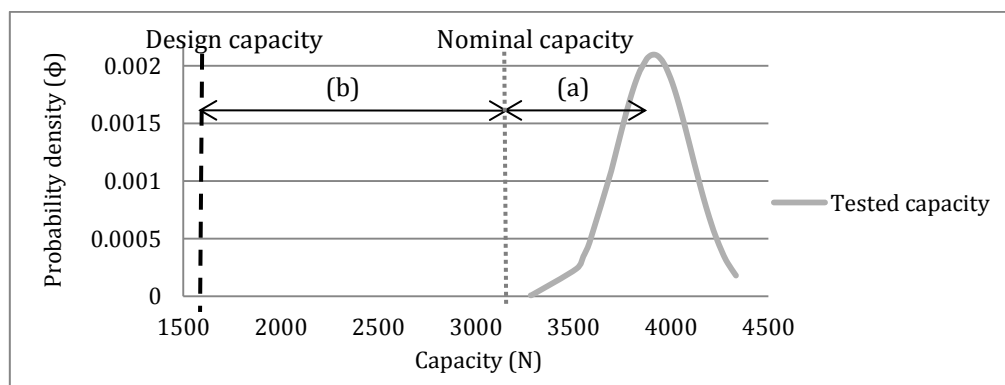


Figure 7.2 Influence of capacity reduction factor



To see how much the capacity reduction factor could be increased without the connection's resistance reliability index attaining a value lower than the target value of 3.6, the exact same analyses as above was performed for a range of capacity reduction factors. Through an increase in the capacity reduction factor, the design capacity increase and thus result in a smaller difference between the design capacity and the actual capacity. The closer the design capacity is to the actual capacity, the smaller the reliability index is. While the same distribution parameters (Table 7.9) and design values (Table 7.10) were used as for the analyses above, the capacity reduction factor was altered to obtain the reduction factor value at which the computed reliability index just exceeds the target value of 3.6. The results of this investigation are given in Figure 7.3.

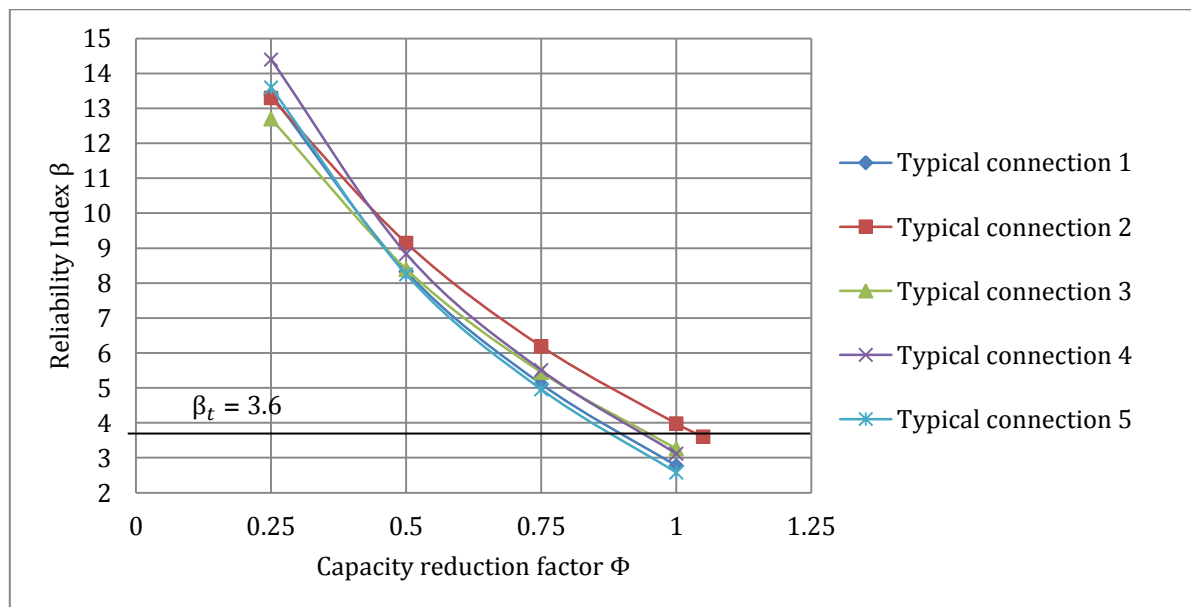


Figure 7.3 Capacity reduction factor vs. computed reliability index

From Figure 7.3 it is clear that a capacity reduction factor of less than one (0.8-0.9) was necessary for all connections except typical connection 2. This implies that the nominal capacity needed to be reduced in order for the connections to be declared reliable. For typical connection 2, however, the capacity reduction factor was increased up to 1.05 and still delivered reliable results. It can thus be said that for the 0.58 mm steel the nominal capacity need not be reduced in order for the connection to be declared reliable.

7.5.4 Results discussion

All of the computed reliability indexes far exceed the target value $\beta_{t,R} = 3.6$, which implies that the design formulation for typical screw connections according to the SANS 10162-2 can be declared sufficiently reliable. More specifically, the formulation was found to be highly



conservative with the reliability indexes exceeded the target value of 3.6 by a considerable margin. The lowest computed index is 8.33 which imply that a significant safety margin seemed to be built in design formulation. It is clear that the computed index values did not differ much for the different steel property sets, with the lowest and highest reliability indexes equal to 8.33 and 9.15 respectively. Now that the reliability status is established, it is worth looking at some informative trends, possible reasons for the significant conservatism and at the relative influence of the variables on the reliability.

In terms of the LSF's used to obtain the reliability measures, the average actual capacities were found to be significantly higher than the predicted design capacities. The more the average actual (tested) capacity exceeds the design capacity, the bigger the reliability margin is. During design, a capacity reduction factor is applied to the nominal connection capacity in order to take material variations and uncertainties into account (Hancock, 2007). SANS suggests a capacity reduction factor of 0.5. The application of this value was identified as the primary cause of the significant conservatism (See Figure 7.2).

The sensitivity factors are indicative of the respective variables' relative importance in the probabilistic assessment. The square of the sensitivity factors indicates the relative contribution of the parameters that were probabilistically modelled. The sensitivity factors show that, as far as the relative contributions to reliability by probabilistically described variables go, the tensile strength and model factor dominate. Theoretically plate thickness and screw diameter may be treated deterministically in further investigations, without introducing significant error in the reliability estimates. As the tensile strength had the highest sensitivity factor (for all steel categories) it is an indication that the tensile strength had the largest influence on the reliability of the parameters probabilistically modelled.

Sensitivity factors depend on the standard deviation and with the tensile strength having the largest spread relative to its mean (CoV of 10%) it is clear why the tensile strength variable was the most influential towards the reliability. The greater the standard deviation of a parameter, the greater the influence of that parameter on the structural reliability.

The typical connection that resulted in the largest reliability index was typical connection 2 (0.58 mm thickness). This was also the connection type of which the nominal capacity could be used without a reduction factor and still be declared sufficiently reliable. The only difference between typical connections 1 and 2 was the steel thickness which allowed for the reliability results of these connections to be compared based on the influence of the thickness. As the thickness have a relatively low sensitivity factor, the difference in thickness did not have a



considerable influence, but the following reductions made to the tensile strength due to the thinness of the steel were the influential factors:

- For 550 MPa grade steel with a thickness less than 0.9 mm, the steel strength used in the design calculations must be taken as the smallest of 495 MPa or 90% of the specified strength value.
- For 550 MPa grade steel with a thickness less than 0.6 mm, the steel strength used in the design calculations must be taken as the smallest of 410 MPa or 75% of the specified strength value.

As both of these typical connections were made of the same steel grade, the tensile strength variable in the actual capacity term of the LSF was the same. Thus, while the actual capacity term remains constant, the design capacity is reduced according to the above mentioned modifications. According to these modifications, the thinner steel is reduced more which implies that the actual capacity would exceed the design capacity by a greater margin for the thinner steel. The more the actual capacity exceeds the design capacity, the more conservative the design capacity is.

Moving away from the field of statistics into a more practical application of the analysis results, the influence of possible changes to the design considerations are deliberated. The design formulation for screwed connections in tilt-and-bearing is a function of the steel thickness, screw diameter, steel tensile strength and the model factor. When a certain capacity is calculated using the design formula, each of these parameters contributes towards this capacity. When a reduction in one of the formulation parameters is made it creates the need for another parameter to be increased in order for this capacity to remain unchanged. As a large safety margin seemed to be built into the design formulation, there is room for increasing the capacity reduction factor and by doing so allow for some of the other parameters to be reduced. This can be made clear in the light of the following example.

Consider a connection that has a design capacity against tilt-and-bearing failure of 10 kN and that is subjected to a load of 8 kN. Say the design capacity of 10 kN was calculated using the design formula given in SANS 10162-2 for typical screwed connections in tilt-and-bearing with a capacity reduction factor of 0.5. By increasing this factor to 0.9, with all other parameters remaining the same, the connection resistance will increase to 18 kN. Seeing that the initial resistance of 10 kN was capable of resisting the applied 8 kN, the new (18 kN) resistance capacity can be reduced to the original 10 kN by reducing some of the connection properties (parameters) that might economically be beneficial to the engineer.



When considering a connection in isolation, it consists of two steel strips (each with a thickness and strength) and a single fastener, thus the connection can be made cheaper by either reducing the steel thickness, the steel strength, or by using a different fastening system. When it comes to the steel thickness and strength, the fact that a connection is a point where members come together, makes it impossible to ignore the members and their purpose in the rest of the structure. Thus, a change in the isolated connection properties can and will most likely have an effect on the members themselves subjected to loadings and failure modes not considered here. In the light of this, the only sensible way where a saving can be achieved is with the fastening system, i.e. by using a thinner screw. However, the economic benefit that can be achieved through this is limited to the saving of insignificant amounts of (screw) material and thus not expected to be worthwhile. Going into further detail regarding an attempt to benefit from the significant safety margin is beyond the scope of this chapter and is recommended for further research.

The target reliabilities are a function of the cost of safety measure and the consequence of failure (JCSS, 2000). If the cost of safety measure is, as it seems to be, primarily related to the screw diameter, the cost to make the structure safer will be low and thus, from a cost perspective, justifies the high reliability index. From the perspective of the consequence of failure, the JCSS probabilistic model code suggests that a structural component with multiple failure modes should be designed for a higher level of reliability. Although it was only tilt-and-bearing failure considered in this research, screwed connections are also designed against shear failure of the screw(s), section tear-out and net section failure of which screw failure is highly brittle and thus needs to be designed for a high level of reliability (JCSS, 2000).

7.5.5 Concluding remark

The SANS 10162-2 was found to be reliable and highly conservative with regards to typical screw connections in tilt-and-bearing. The lowest computed reliability index is equal to 8.33, which implies that all computed indexes exceeded the target value of 3.6 by a considerable margin. The large capacity reduction factor of 0.5 was identified as the main reason for the significant safety margin between the computed indexes and the target values. Further research is recommended to investigate whether some sort of modification can be made to the design considerations in attempt to beneficially utilise the significant safety margin.

Chapter 8

Conclusions and recommendations

8.1 Introduction

The main objectives of this research were to investigate the resistance reliability of SANS 10162-2 with regards to typical screw connections and to determine a suitable model factor for the prediction model of such connections. Screwed connections have a range of possible failure modes of which this thesis investigated the combined failure mode of screw tilting and hole bearing. Through a rational assessment of uncertainties, the reliability status of typical screwed connections was analysed following a probabilistic approach. Through the use of experimental work and the design code resistance prediction model, the model accuracy was determined and quantified in the form of a model factor.

The experimental work was discussed in Chapter 5 which was used for the model factor determination in Chapter 6 followed by the reliability analyses in Chapter 7. These investigations were supplemented by important literature in Chapters 2-4. The current chapter concludes this document with a summary of the main research findings and recommendations for future research.

For Referencing purposes, Table 8.1 provides the properties of the typical connections and steel property sets used for the model factor determination and reliability analyses.

Table 8.1 Typical connections and steel property sets

Typical connection	Steel property set	Number of steel coils	Steel thickness (mm)	Steel grade (MPa)	Connector type
1	1	4	0.8	550	screw
2	2	1	0.58	550	screw
3	3	1	1.0	300	screw
4	4	1	0.8	230	screw
5	1	1	0.8	550	Pop-rivet



8.2 Conclusions

8.2.1 Model factor determination

The model factors were determined as the ratio of the tested capacities over the capacities predicted using the SANS 10162-2 design formulation for tilt-and-bearing. The main findings from the experimental work used for the model factor determination and results are:

- Single point screwed connections in cold-formed steel (CFS) structures are most likely to fail in a combination of screw tilting and hole bearing when the connected members are 1.0 mm thick or thinner and the end distance not less than 1.5 times the screw diameter.
- Riveted connections are most likely to fail in tilting and bearing, but undergo significantly less bearing than that of screwed connections. Tilting is the dominant factor for both screwed and riveted single-point connections.
- After testing two different screw threads during the preliminary tests, it was found that the screw thread only had an effect on the force-displacement curve, but not on the ultimate connection capacity.
- In the determination of the cross sectional area of the specimens used for the tensile tests, it was found that the original cross sectional area is disturbed by necking and localised elongation during the test. It is therefore necessary to measure the area prior to testing. Regarding the hydrochloric acid used to remove the coating, it was found that the acid did not affect the steel strength when the coating is removed prior to testing.
- The standard test for single-point fastener connections in Appendix F of the SANS 10162-2 is an adequate testing procedure for determining the actual connection capacities.
- After comparing the predicted failure modes and capacities with those observed from the experimental work it was concluded that Section 5 of the SANS 10162-2 is capable of accurately predicting both the failure mode and the connection capacity of typical screw connections. The average model factor for all typical connections (Table 8.1) exceeded unity.
- Model factors tend to increase for an increase in the thickness and decrease for an increase in the steel strength.
- The more ductile 230 MPa steel grade by far delivered the most conservative results with an average model factor as high as 1.25. For the 550 MPa grade steel, the prediction model had a negligible conservative bias with the average model factor equal to 1.01.



- The lack of conservatism of the prediction model when using 550 MPa grade steel seems to be taken into account by SANS 10162-2 during the design stages through a reduction in the nominal tensile strength, in that Clause 1.5.1.4 recommends specific reductions for steel thicknesses less than 0.6 mm and 0.9 mm.
- Both the standard deviations and the coefficients of variation of the model factors were small for all the typical connections. This implies a repeatable testing procedure as well as a good prediction model.

8.2.2 Reliability analysis

The main findings from the reliability analysis are:

- Connections designed according to Section 5.4 of the SANS 10162-2 were found to be reliable across the range of material strength and plate thicknesses. A considerable safety margin seemed to be incorporated in the design considerations with all computed resistance reliability indexes far exceeding the target value of 3.6.
- It is clear that the computed index values did not differ much for the different typical connections, with the lowest and highest reliability indexes equal to 8.33 and 9.15 respectively.
- A high reliability index is an indication of a design value adequately less than the actual capacity with the analysed limit state function defined as the difference between the actual capacity and the design capacity. It was therefore concluded that the main contributor towards the high reliability indexes and the significant conservatism was the large design capacity reduction factor of 0.5 suggested by SANS.
- The FORM sensitivity factors are indicative of the respective variables' relative importance in the probabilistic assessment. The square of the sensitivity factors tells the relative contribution. Of the parameters probabilistically modelled, the thickness and nominal diameter had the lowest sensitivity factors.
- Theoretically plate thickness and screw diameter may be treated deterministically in further investigations, without introducing significant error in the reliability estimates.
- The only difference between steel property sets 1 and 2 was the thickness. This allowed the results comparison of these two property sets to be based on the influence of the thickness on the reliability. Although the thickness had relatively small sensitivity factors, the thickness, through design considerations, indirectly influenced the reliability. According to Section 1.5.1.4 of the SANS 10162-2 the steel strength, when using 550 MPa grade steel, must be reduced by 75% and 90% for thicknesses less than



0.6 mm and 0.9 mm respectively. This results in lower design capacity estimates and thus increases reliability.

- Of the parameters probabilistically modelled, the tensile strength was the dominant contributor towards the structural reliability. For the 1 mm thick steel, the tensile strength was highly dominant with a squared sensitivity factor four times that of the squared sensitivity factor associated with the model factor. For the other steel property sets, the model factor was more influential, but was still less than the influence of the tensile strength.
- The tensile strength's dominant contribution towards reliability was mainly due to a more widely spread set of data around the mean as sensitivity factors is dependent on the standard deviation.
- The model factor's influence on the reliability was dependent on the model factor's standard deviation. The coefficient of variation of the model factor differed among the different typical connections and was found to increase the model factor sensitivity factor when increased.
- After it was established that capacities of screwed connections are predicted (designed) with a significant amount of conservatism it was argued that the safety margin may be reduced by adjusting the capacity reduction factor to a value just ensuring reliability (i.e. the computed reliability index just exceeds the target index). The capacity reduction factors necessary for the computed reliability indexes to just exceed the target value ranged between 0.9 and 1.0 for most of the steel properties. For the 0.58 mm steel, a capacity reduction factor of up to 1.05 still delivered reliable results. This implies that the nominal capacity of a connection consisting of 0.58 mm steel members needed not to be reduced.
- The design formulation for screwed connections in tilt-and-bearing is a function of the thickness, screw diameter, tensile strength and the capacity reduction factor. When a certain capacity is calculated using this formula, each of these parameters contributes towards this capacity. By reducing one of these parameters it creates the need for another parameter in the formulation to be increased, for this capacity to remain unchanged. In a more practical application of the results it can be argued that an increase in the capacity reduction factor allows for a reduction in some of the other parameters (thickness, steel strength) and thus possibly result in some economic benefits.
- When considering a connection in isolation, it consists of two steel strips (each with a thickness and strength) and a single fastener, thus the connection can be made cheaper



by either reducing the thickness, the strength or use a different fastening system. When it comes to the steel thickness and strength, the fact that a connection is a point where members come together, makes it impossible to ignore the members and their purpose in the rest of the structure. Thus, a change in the isolated connection properties can and will most likely have an effect on the members themselves subjected to loadings and failure modes not considered here. In the light of this, the only sensible way where a saving can be achieved is with the fastening system, i.e. by using a thinner screw. However, the economic benefit that can be achieved through this is limited to the saving of insignificant amounts of (screw) material and thus not expected to be worthwhile. Going into further detail regarding an attempt to benefit from the significant safety margin is beyond the scope of this chapter and is recommended for further research.

8.3 Recommendations

As all experiments and analyses were done according to well-studied standard procedures, no real limitations were experienced regarding the methods followed. There was, however, some limitations regarding the materials used of which recommendations are given here on how to overcome these limitations. This section is concluded with recommendations for future works that holds some relevance to the current research.

The primary limitation during the reliability analysis was the lack of measurements from different steel coils. Three of the four steel property sets were represented through a single coil. It is strongly recommended that at least four coils, but preferably more per steel property set are used for future relevant work. By using such a diverse set of data, the variable parameters would be more representative of the wide range of materials used in practice.

All connection test specimens made use of a single 4.8 mm screw for connecting the members. As the same screw diameter was used no observations could be made regarding the influence of different screw diameters on the neither the model factor nor the reliability. The SANS allows for a screw diameter range of 3 mm to 7 mm and it is recommended that different (smaller and bigger than 4.8 mm) screw diameters are used.

Regarding future works that would include the same sort of experiments done for the purpose of the current research, the test procedures used here are strongly recommended. Both the tensile test and connection test methods used for this research were found adequate and reliable as it delivered accurate results. The tensile testing was done according to the SANS 6892 and the connection tests were based on Appendix F of the SANS 10162-2.



As the prediction model for typical screwed connections was found to be highly conservative, an economic way of beneficially utilising the significant safety margin was briefly introduced in the previous section. From a reliability point of view, capacity reduction factors can be increased from 0.5 to 0.9 and still deliver reliable results. This allows for a reduction in either the steel strength or perhaps the steel thickness when considering that a certain capacity must be retained. It is recommended that a more detailed study is done regarding these possible design considerations modifications. Whether it would be economically beneficial and if the reduction in cost is of considerable value are some points to look at. The connection in this research was considered as an isolated structural component, but it is advised that the system and its reliability are also considered.

8.4 Concluding statement

The prediction model for typical screwed connections given in the SANS 10162-2 is capable of accurately predicting the connection capacity. The design provisions for these connections ensure an adequate (even high) level of conservatism. A number of typical connection configurations with different plate thicknesses and steel strength achieved resistance reliability index values of $\beta > 8$. The application of a capacity reduction factor of $\phi = 0.5$ to the nominal capacity proved to be the main contributor to conservatism. There may be scope to increase this factor to obtain reliability closer to the target of $\beta_t = 3.6$ if economic benefit can be derived from such a calibration.



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Appendices



Appendix A

Experimental data

The following selected experimental data is presented in this appendix:

Tensile testing (A.1)

- Cross sectional measurements
- Tensile forces (force-displacement relationships)
- Tensile stresses (strain-strain results)

Connection testing (A.2)

- Connection capacity test results for all typical connections (force-displacement relationships)

Model factor determination (A.3)

- Tested capacities
- Predicted capacities
- Determined model factors



A.1 Tensile testing data

A.1.1 Cross sectional measurements

Table A.1 Test specimen cross sections

Steel property set (steel properties)	Specimen	Thickness (mm)					Width (mm)					Area (mm^2)		
		t1	t2	t3	t4	t5	tave	w1	w2	w3	w4	w5	wave	Average area
1 (0.8 mm, 550 MPa, Coil 1)	1	0.775	0.77	0.77	0.77	0.77	0.771	20	20	20	20.01	19.99	20.00	15.42
	2	0.77	0.77	0.77	0.77	0.77	0.770	20.03	20.04	20.08	20	20.06	20.04	15.43
	3	0.765	0.77	0.77	0.77	0.77	0.769	20.01	20	20.02	20.04	20.01	20.02	15.39
	4	0.765	0.77	0.765	0.765	0.765	0.766	20.06	20.07	20.05	20.08	20.02	20.06	15.36
	5	0.77	0.77	0.765	0.765	0.77	0.768	20	19.98	20.03	20.01	20.02	20.01	15.37
	6	0.77	0.77	0.77	0.765	0.77	0.769	20.02	19.97	20.02	20.02	20.01	20.01	15.39
1 (0.8 mm, 550 MPa, Coil 2)	7	0.77	0.77	0.77	0.77	0.77	0.770	20.20	20.18	20.17	20.22	20.17	20.19	15.54
1 (0.8 mm, 550 MPa, Coil 3)	8	0.77	0.77	0.77	0.77	0.77	0.767	20.23	20.26	20.20	20.23	20.24	20.23	15.52
1 (0.8 mm, 550 MPa, Coil 4)	9	0.76	0.76	0.76	0.75	0.75	0.753	20.07	20.08	20.08	20.08	20.08	20.08	15.12
1 (0.8 mm, 550 MPa, Coil 4)	10	0.76	0.76	0.76	0.75	0.75	0.754	20.07	20.03	19.95	19.98	19.94	19.99	15.08
	11	0.77	0.77	0.77	0.77	0.76	0.764	20.10	20.13	20.12	20.16	20.10	20.12	15.37
2 (0.58 mm, 550 MPa) (single coil)	12	0.77	0.77	0.77	0.77	0.76	0.764	20.09	20.09	20.10	20.13	20.10	20.10	15.36
	1	0.48	0.48	0.48	0.48	0.48	0.477	19.99	19.99	20.00	19.92	19.87	19.95	9.47
3 (1.0 mm, 300 MPa) (single coil)	2	0.48	0.48	0.47	0.47	0.47	0.473	19.86	19.84	19.85	19.95	20.02	19.90	9.41
	1	0.96	0.96	0.96	0.96	0.96	0.960	20.02	20.02	20.02	20.20	19.98	20.05	19.25
	2	0.96	0.96	0.96	0.96	0.96	0.955	19.93	19.91	19.89	19.85	19.86	19.89	19.08
4 (0.8 mm, 230 MPa) (single coil)	3	0.96	0.96	0.95	0.95	0.96	0.953	19.92	20.00	19.92	19.93	19.95	19.94	19.01
	1	0.76	0.76	0.77	0.76	0.77	0.763	19.92	19.92	19.92	19.92	19.92	19.92	15.20
	2	0.76	0.76	0.76	0.76	0.76	0.760	19.87	19.90	19.88	19.89	19.86	19.88	15.11
	3	0.76	0.76	0.76	0.75	0.75	0.753	19.83	19.81	19.83	19.81	19.84	19.82	14.93
15.39	4	0.76	0.76	0.76	0.76	0.76	0.759	19.86	19.84	19.85	19.86	19.88	19.86	15.07
	15.08													



A.1.2 Tensile Forces

A.1.2.1 Force-displacement relationships

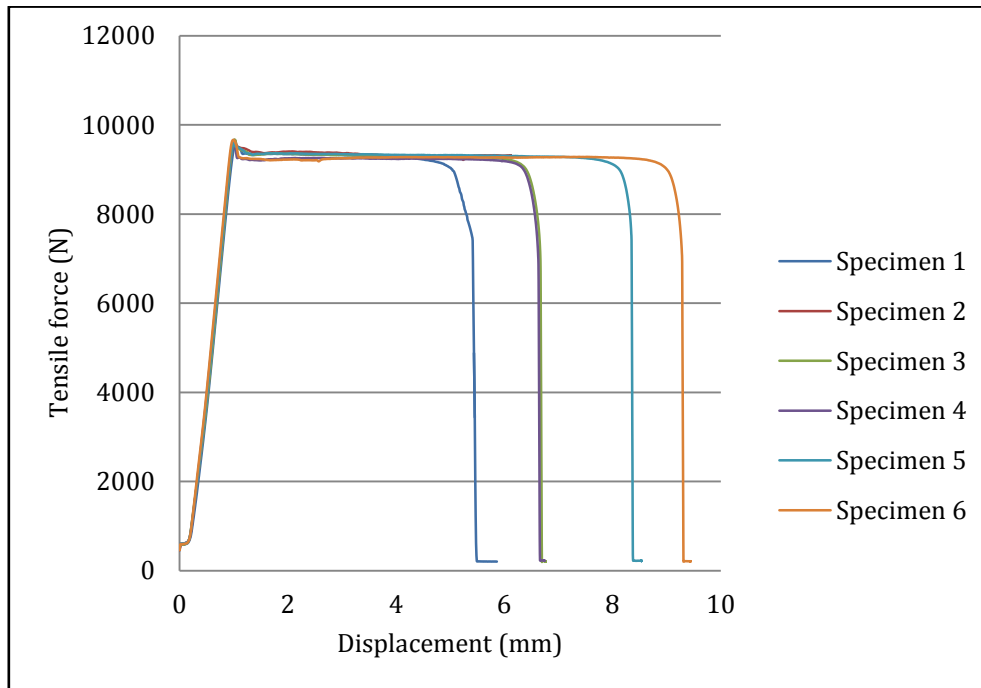


Figure A.1 Steel property set 1 (0.8 mm, 550 MPa, coil 1)

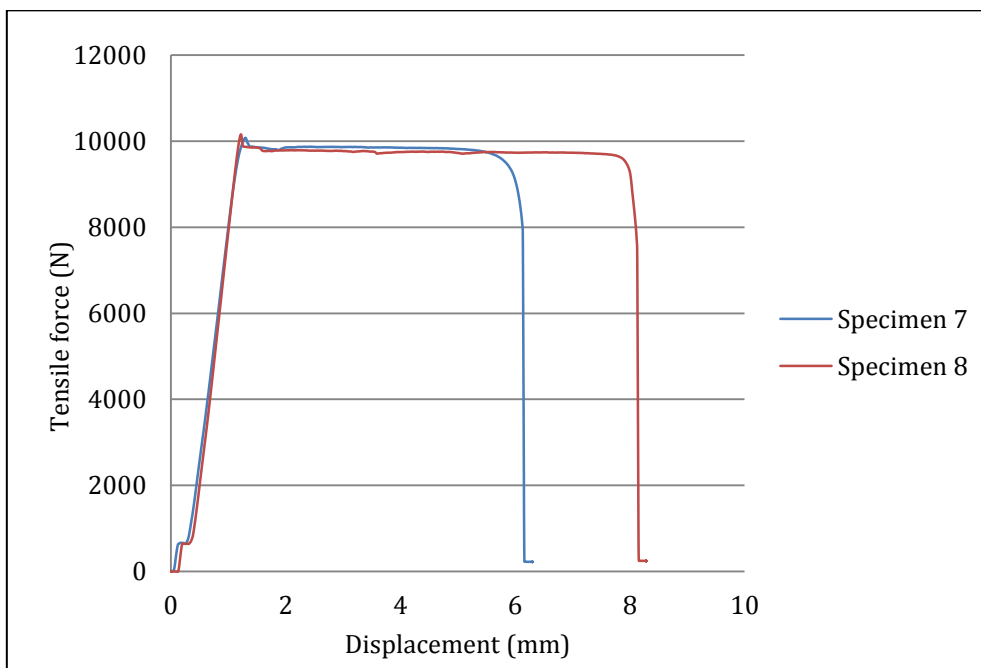


Figure A.2 Steel property set 1 (0.8 mm, 550 MPa, coil 2)

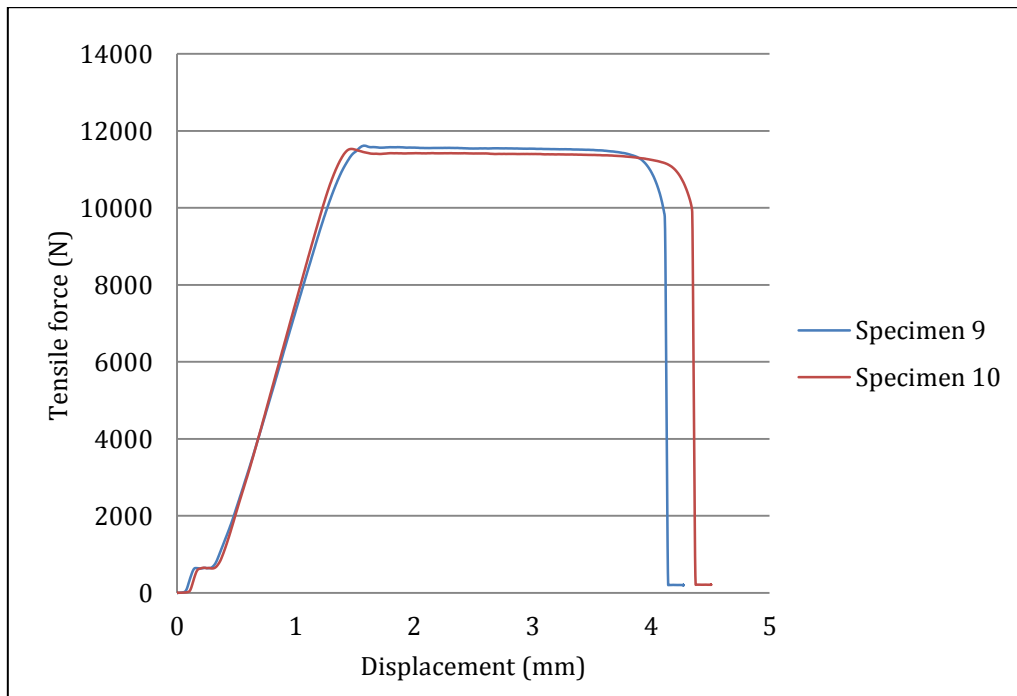


Figure A.3 Steel property set 1 (0.8 mm, 550 MPa, coil 3)

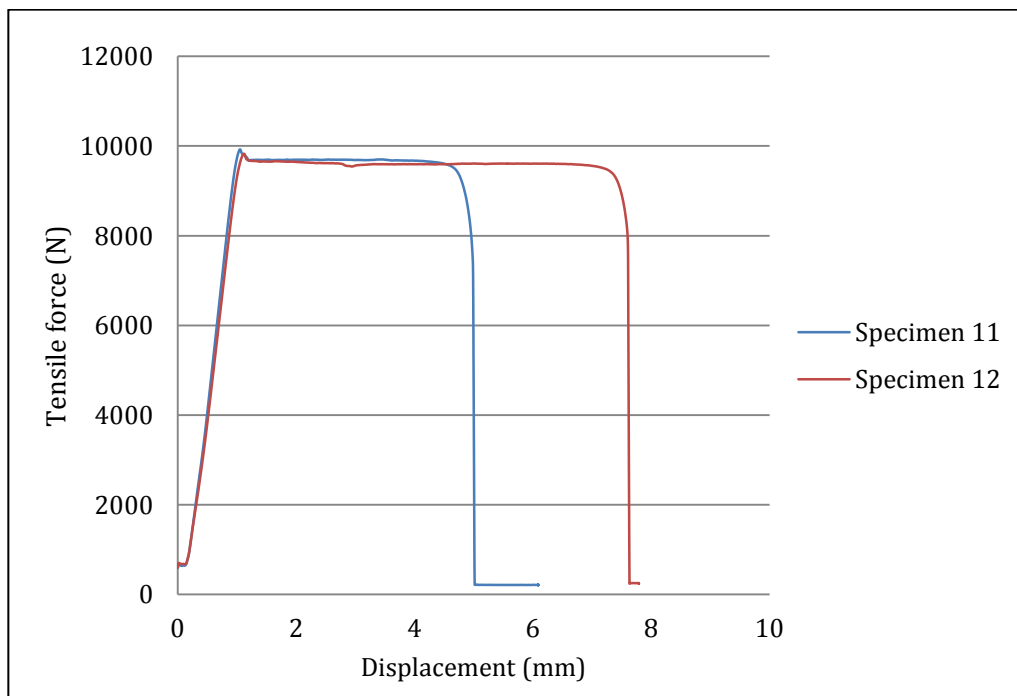


Figure A.4 Steel property set 1 (0.8 mm, 550 MPa, coil 4)

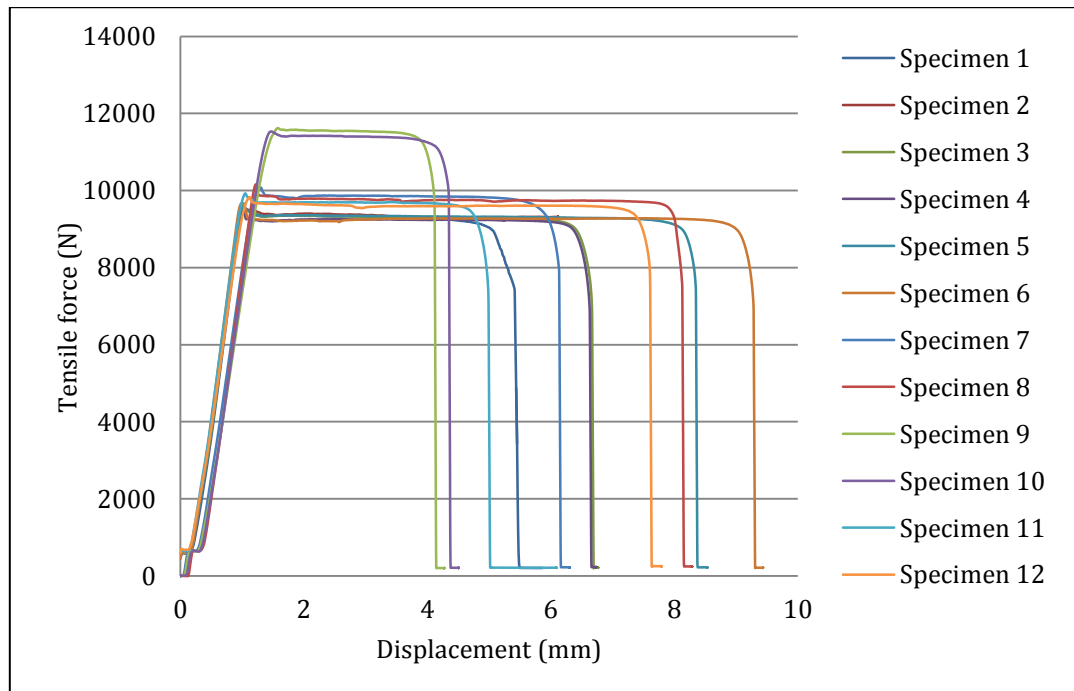


Figure A.5 Steel property set 1 (0.8 mm, 550 MPa, all coils)

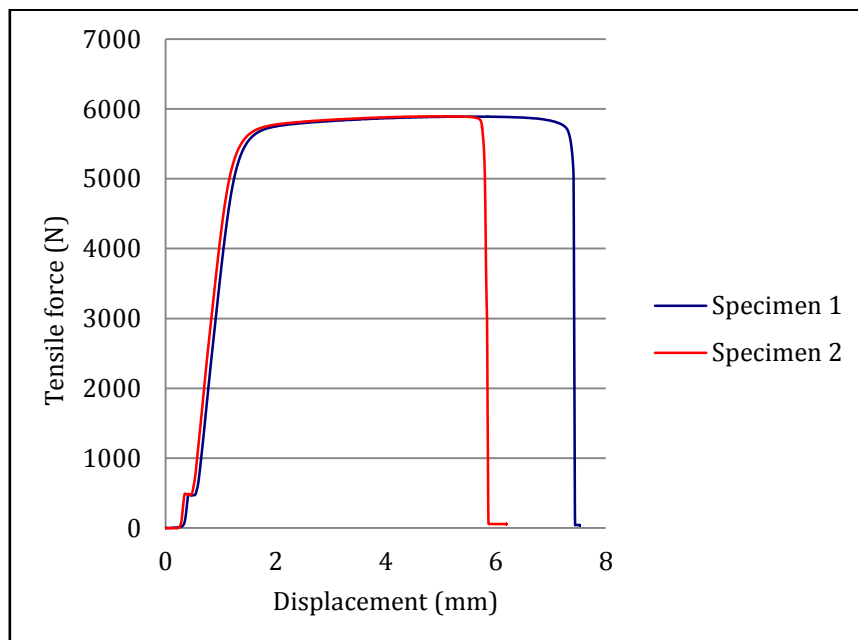


Figure A.6 Steel property set 2 (0.58 mm, 550 MPa)

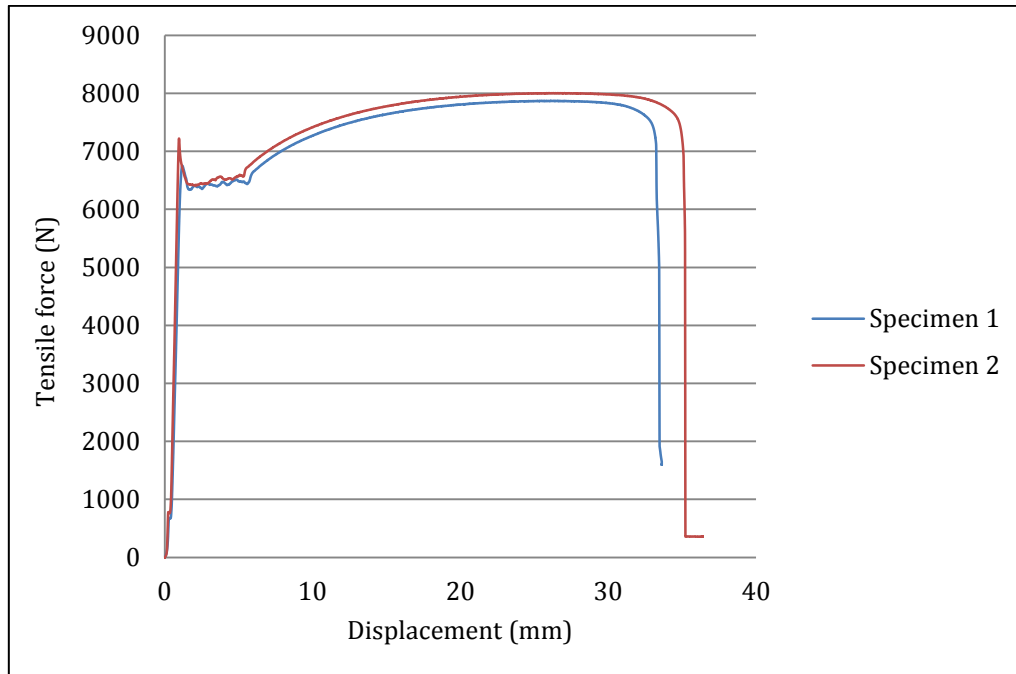


Figure A.7 Steel property set 3 (1.0 mm, 300 MPa)

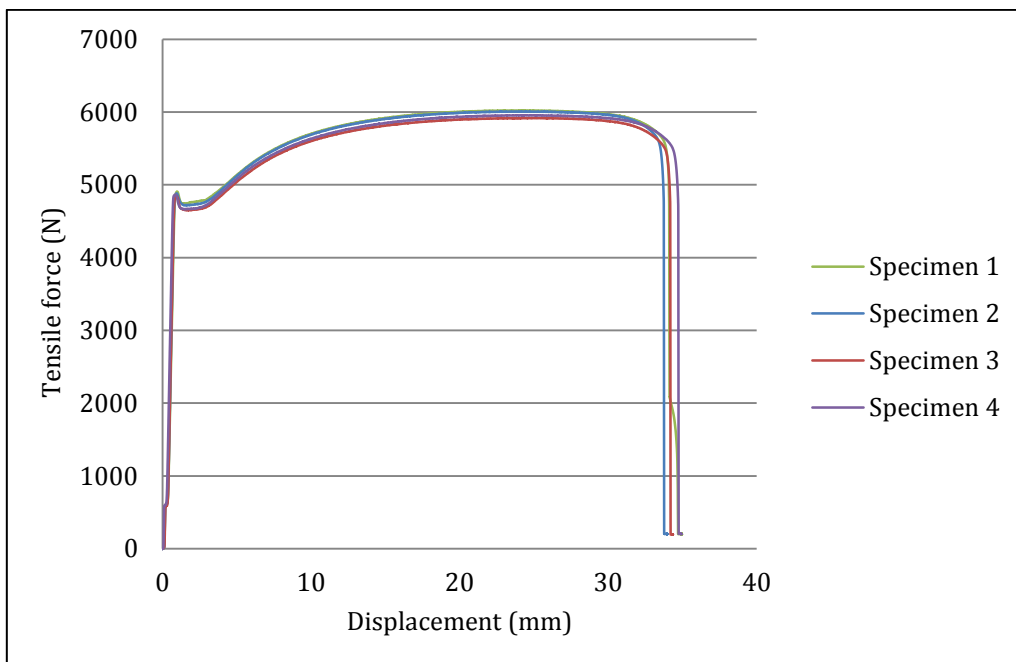


Figure A.8 Steel property set 4 (0.8 mm, 230MPa)



A.1.2.2 Maximum tensile force results

Table A.2 Tested Tensile forces

Steel property set (steel properties)	Specimen	Force (N)	
		F	
1 (0.8 mm, 550 MPa, Coil 1)	1	9363.85	9324.75
	2	9403.81	
	3	9276.29	
	4	9258.44	
	5	9363.85	
	6	9282.24	
1 (0.8 mm, 550 MPa, Coil 2)	7	9872.19	9832.22
	8	9792.24	
1 (0.8 mm, 550 MPa, Coil 3)	9	11579.65	11501.01
	10	11422.36	
1 (0.8 mm, 550 MPa, Coil 4)	11	9701.02	9680.82
	12	9660.62	
2 (0.58 mm, 550 MPa) (single coil)	1	5890.33	5892.46
	2	5894.58	
3 (1.0 mm, 300 MPa) (single coil)	1	8042.33	7971.73
	2	7869.67	
	3	8003.20	
4 (0.8 mm, 230 MPa) (single coil)	1	6023.75	5976.86
	2	6008.86	
	3	5917.85	
	4	5956.98	



A.1.3 Tensile stresses

A.1.3.1 Stress-strain relationships

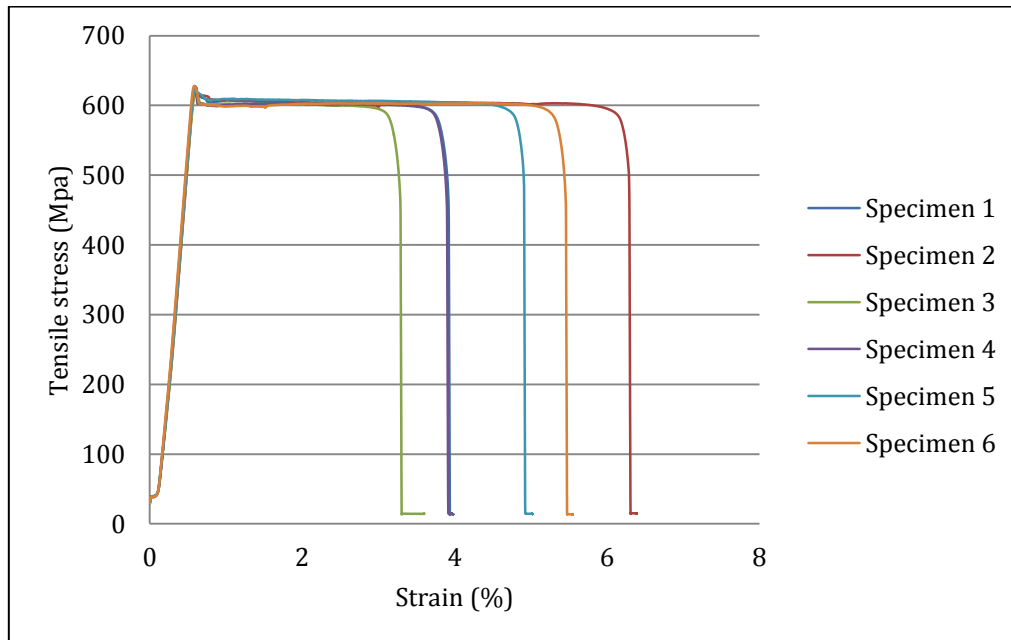


Figure A.9 Steel property set 1 (0.8 mm, 550 MPa, coil 1)

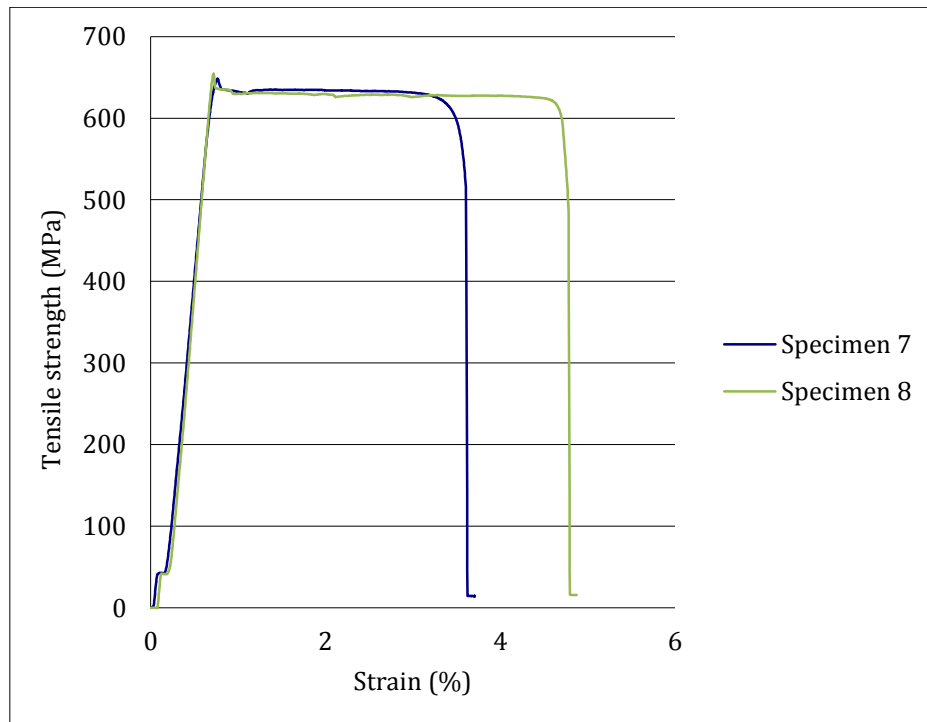


Figure A.10 Steel property set 1 (0.8 mm, 550 MPa, coil 2)

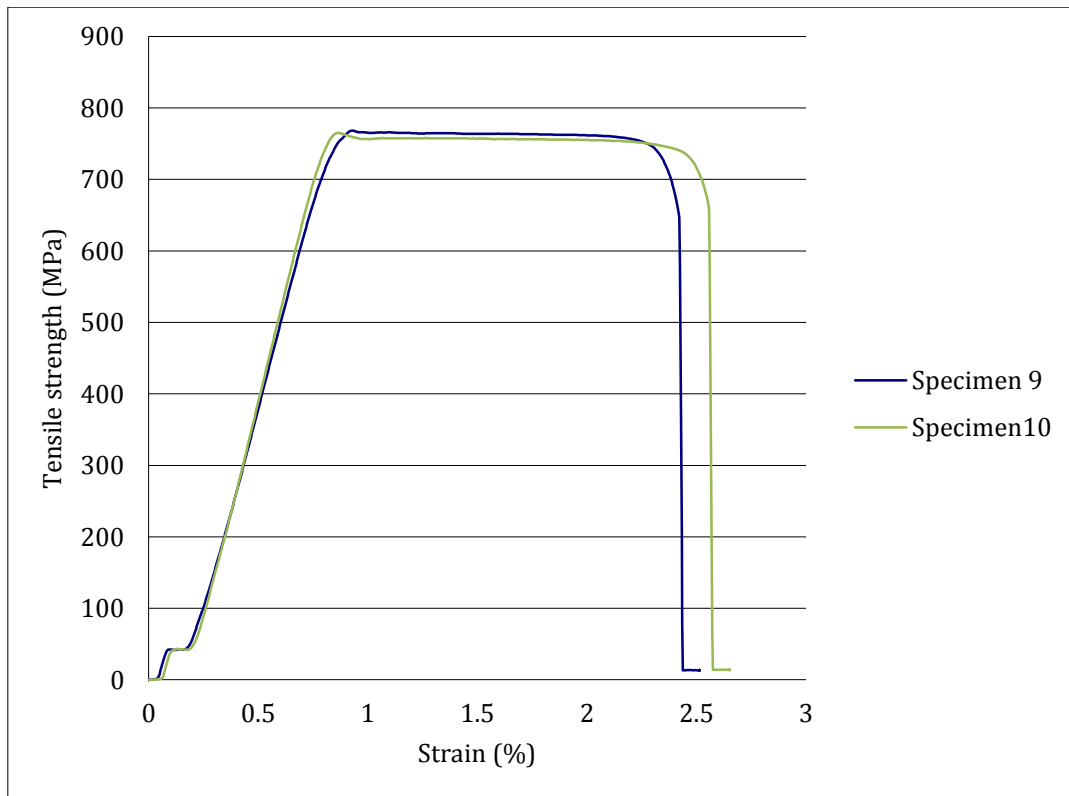


Figure A.11 Steel property set 1 (0.8 mm, 550 MPa, coil 3)

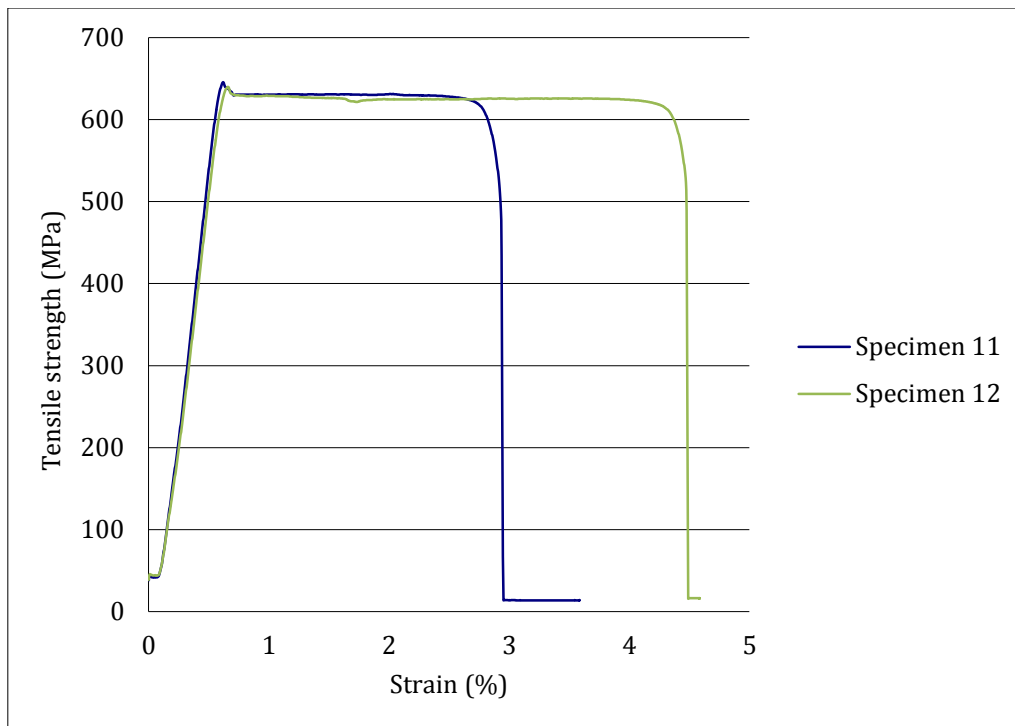


Figure A.12 Steel property set 1 (0.8 mm, 550 MPa, coil 3)

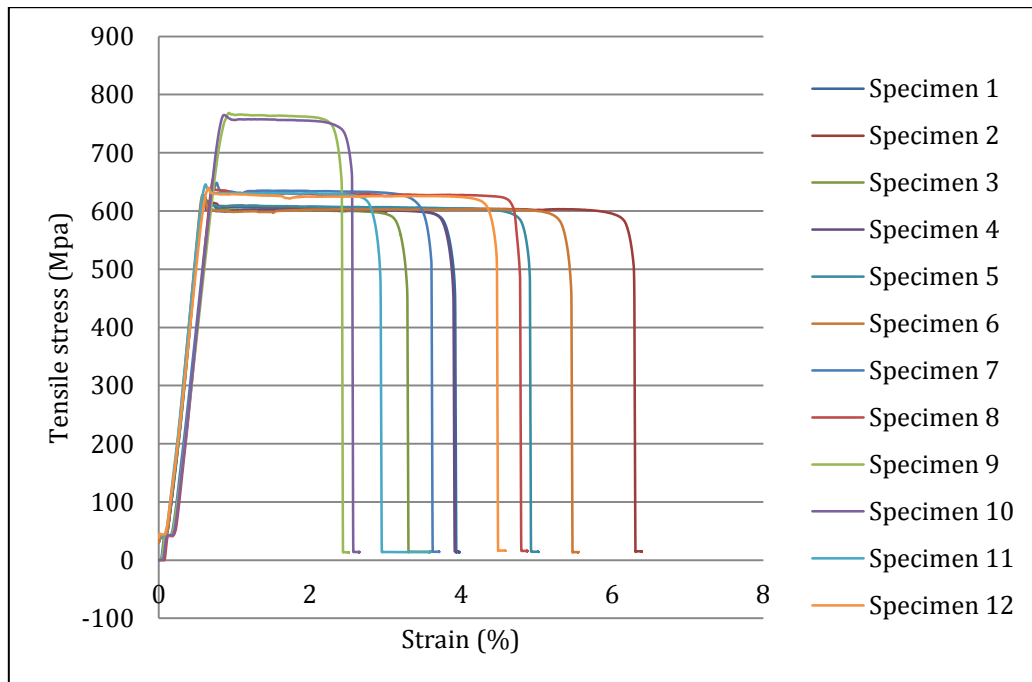


Figure A.13 Steel property set 1 (0.8 mm, 550 MPa, all coils)

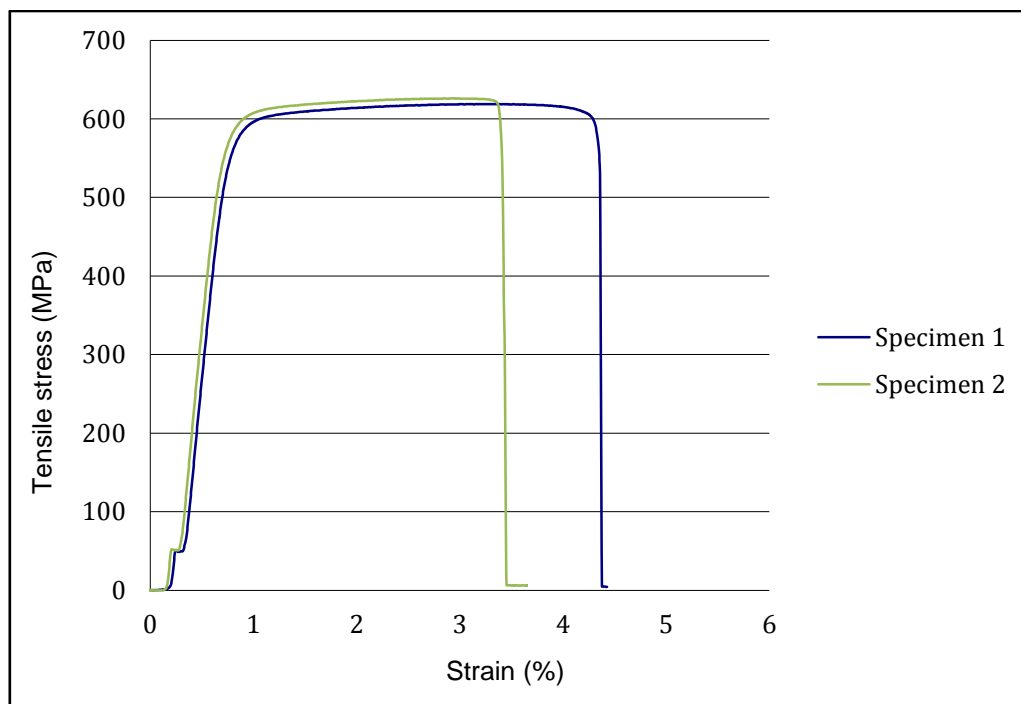


Figure A.14 Steel property set 2 (0.58 mm, 550 MPa)

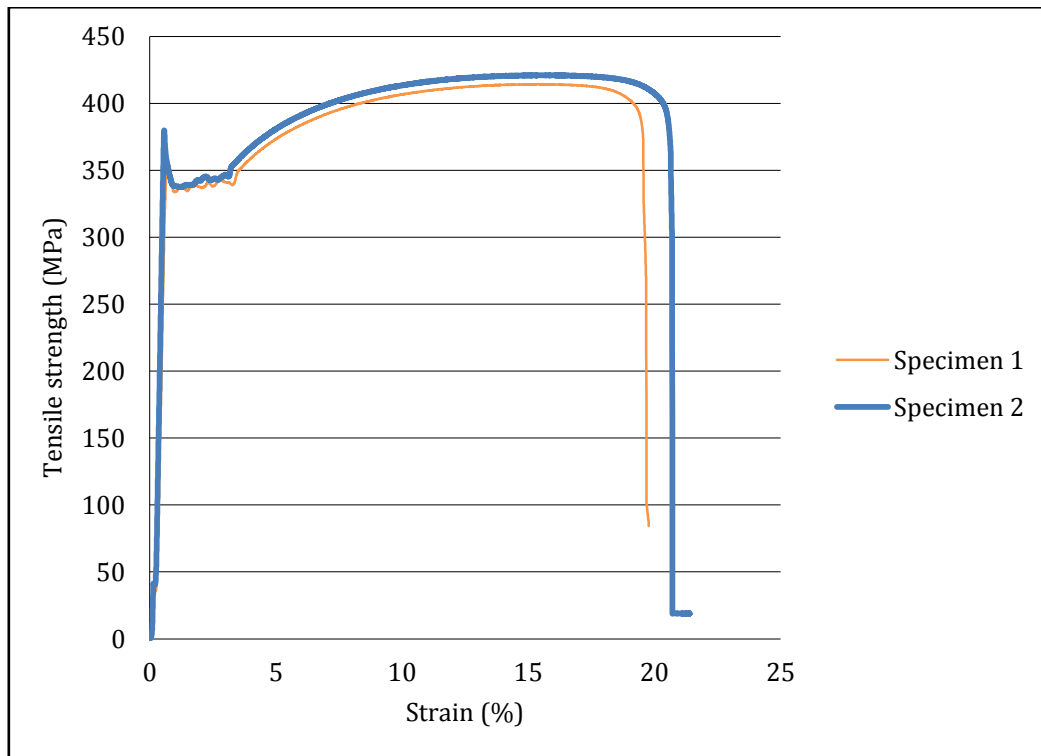


Figure A.15 Steel property set 3 (1.0 mm, 300 MPa)

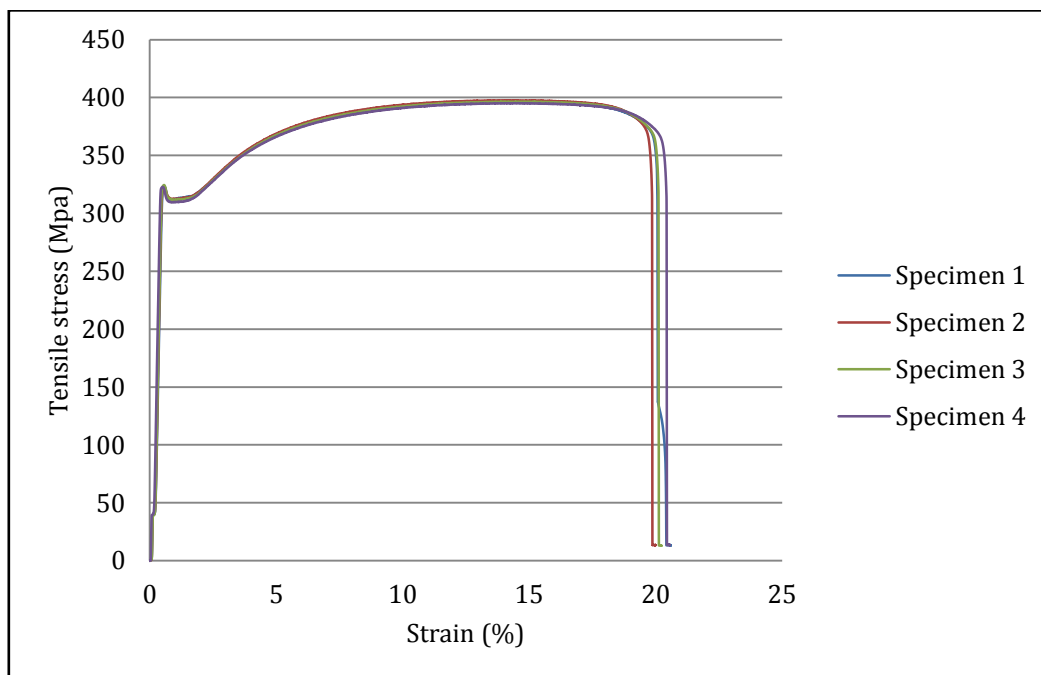


Figure A.16 Steel property set 4 (0.8 mm, 230MPa)



A.1.3.2 Ultimate tensile stress results

Table A.3 Tested tensile stresses

Steel property set (steel properties)	Tensile strength(Mpa)		
	Specimen	Fu	Average Fu
1 (0.8 mm, 550 MPa, Coil 1)	1	607.25	605.76
	3	609.36	
	4	602.66	
	6	602.65	
	7	609.38	
	9	603.29	
1 (0.8 mm, 550 MPa, Coil 2)	1	635.08	633.05
	3	631.03	
1 (0.8 mm, 550 MPa, Coil 3)	1	765.91	761.80
	3	757.68	
1 (0.8 mm, 550 MPa, Coil 4)	1	631.03	630.03
	3	629.03	
2 (0.58 mm, 550 MPa) (single coil)	1	618.86	622.49
	3	626.11	
3 (1.0 mm, 300 MPa) (single coil)	2	417.87	417.76
	3	414.34	
	4	421.07	
4 (0.8 mm, 230 MPa) (single coil)	1	396.33	396.43
	2	397.71	
	3	396.44	
	4	395.23	



A.2 Connection testing data

A.2.1 Typical connection 1 (Screwed, 0.8 mm, 550 MPa, 4 Coils, 132 Specimen)

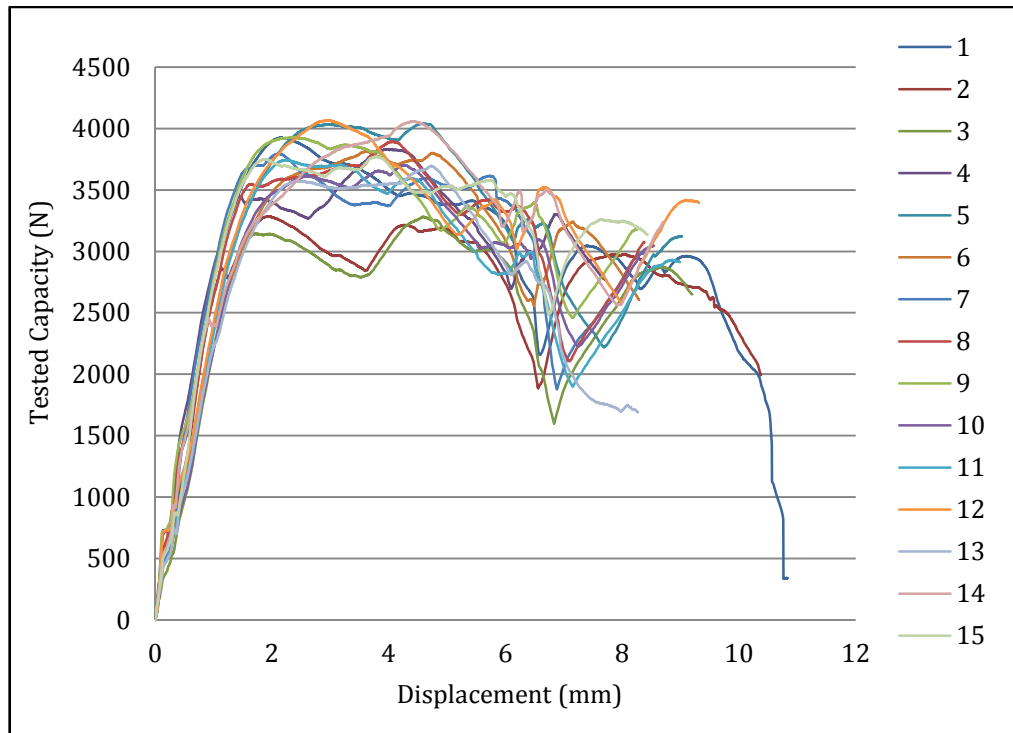


Figure A.17 Coil1, Specimen 1-15

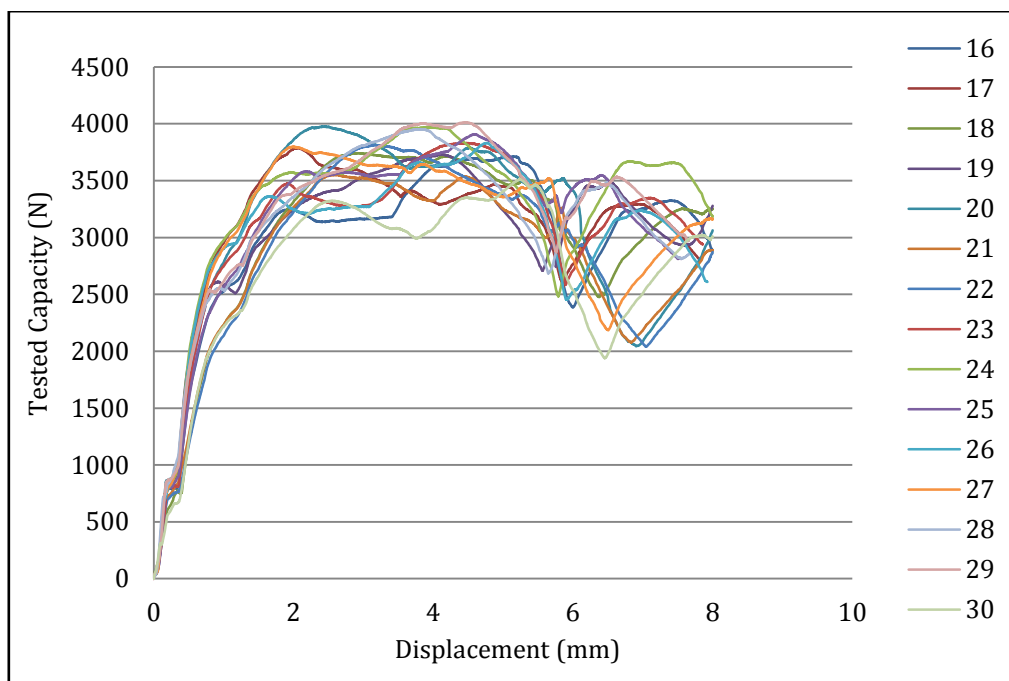


Figure A.18 Coil 1, Specimen 16-30

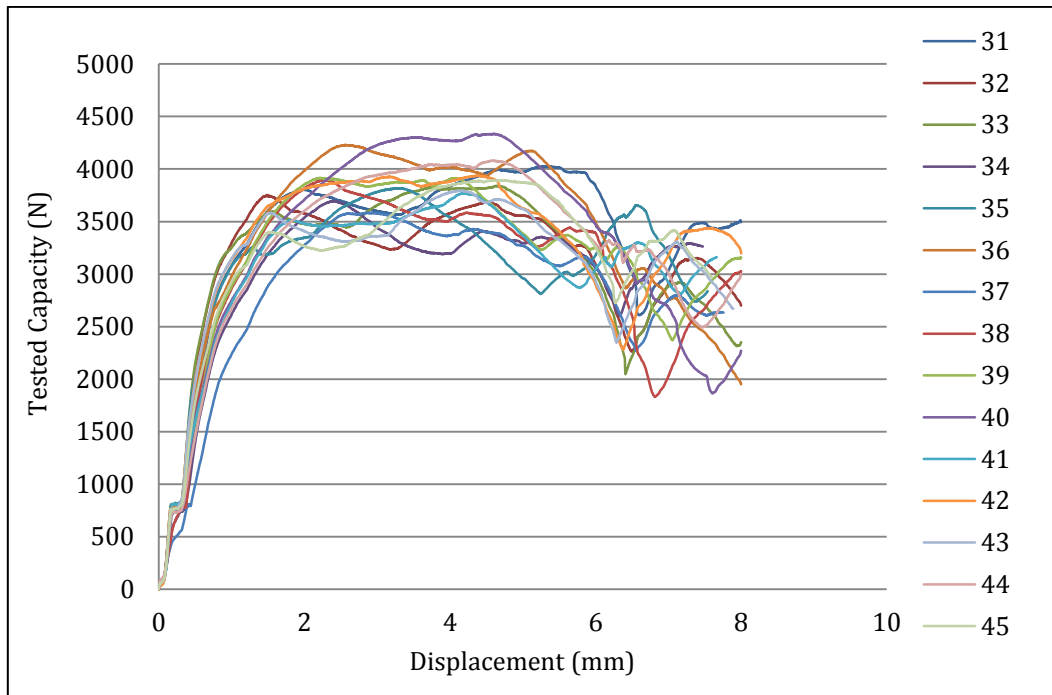


Figure A.19 Coil 1, Specimen 31-45

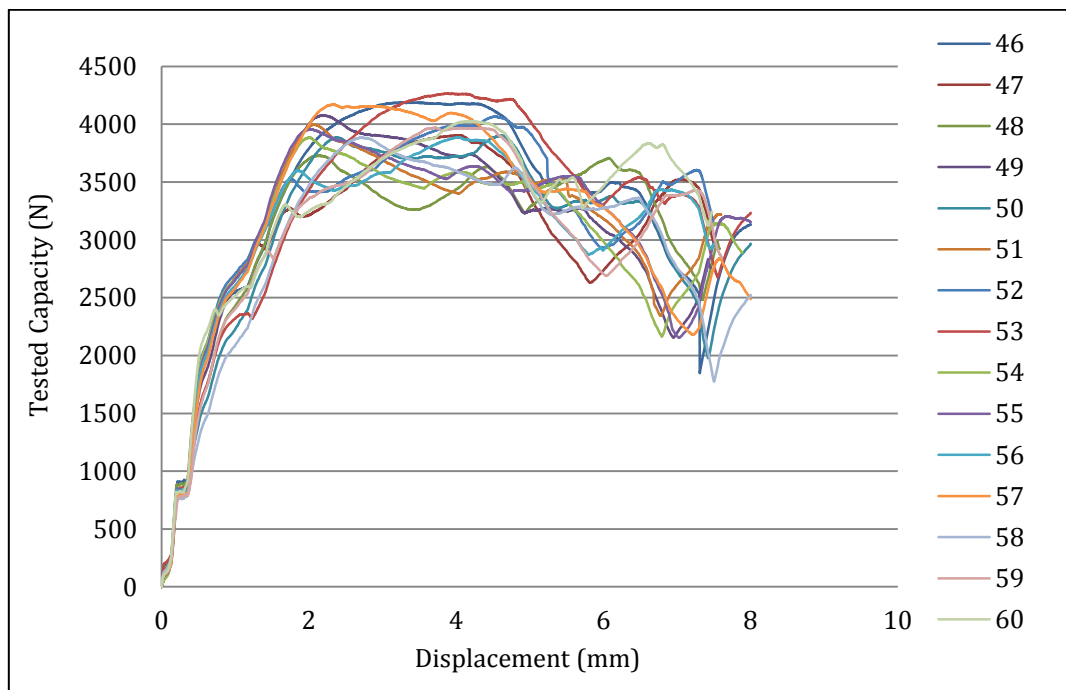


Figure A.20 Coil 1, Specimen 46-60

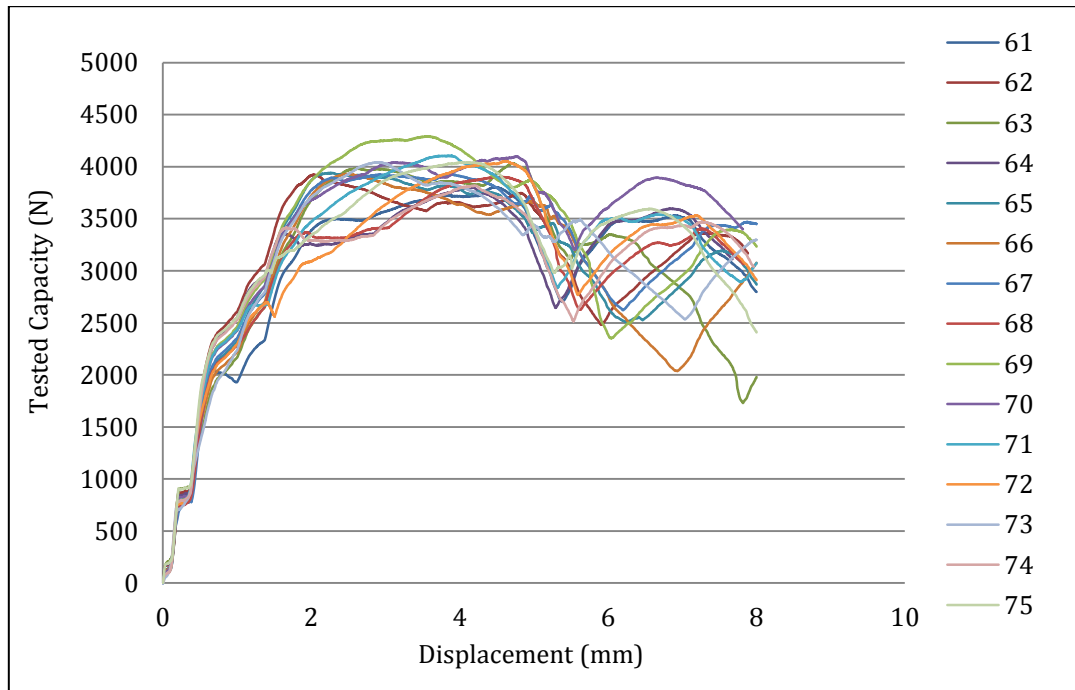


Figure A.21 Coil 1, Specimen 61-75

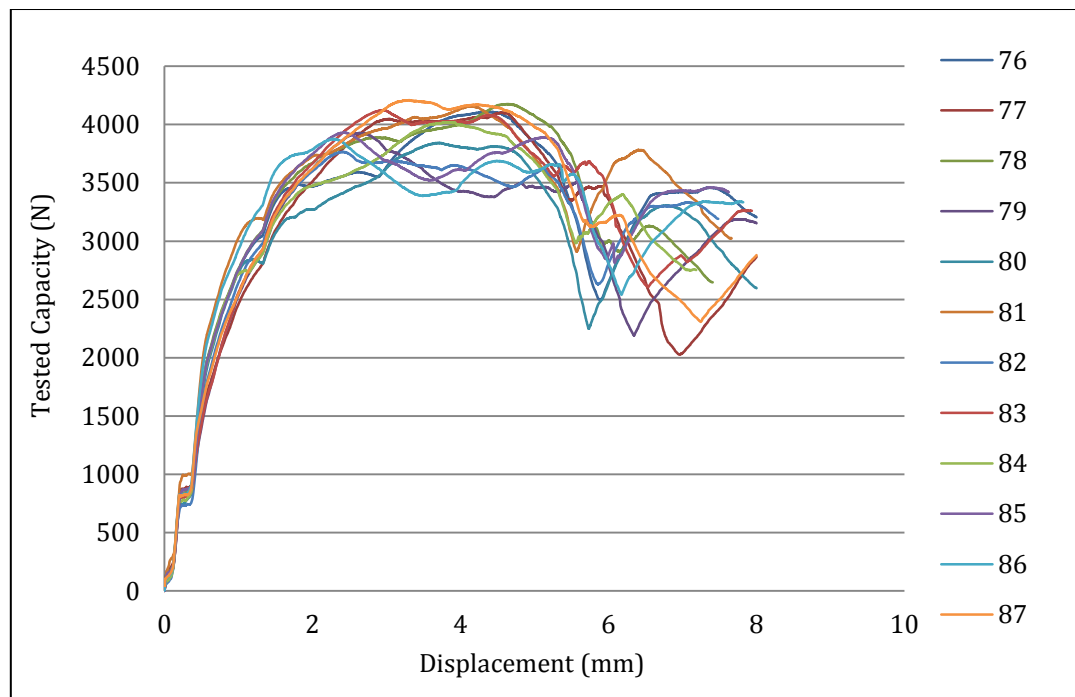


Figure A.22 Coil 1, Specimen 76-87

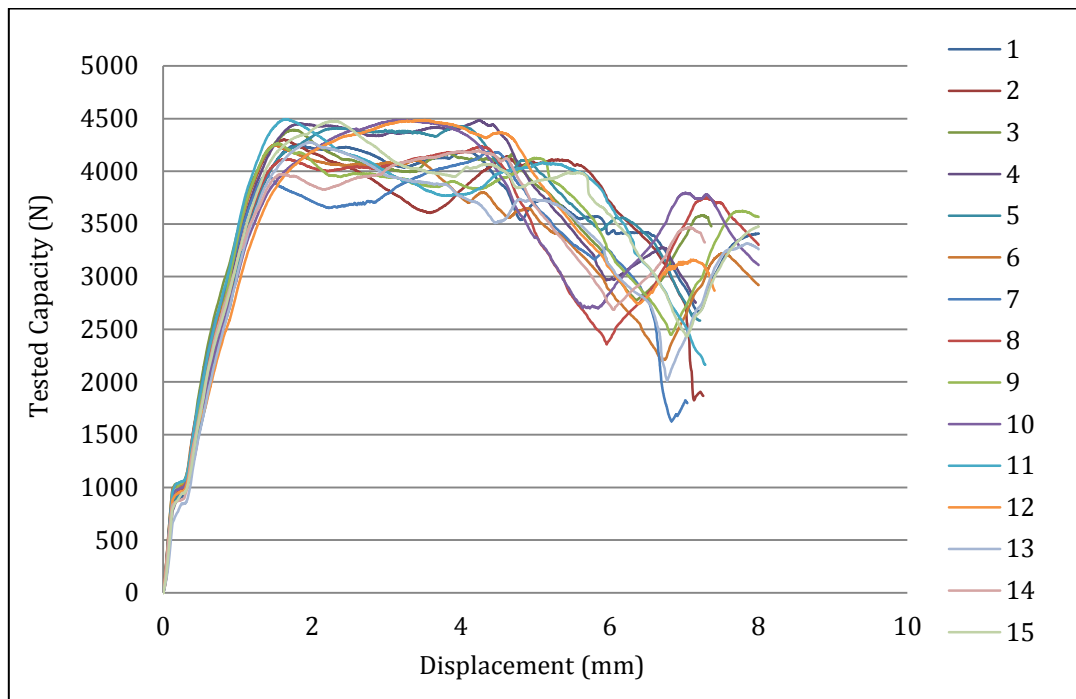


Figure A.23 Coil 2, Specimen 1-15

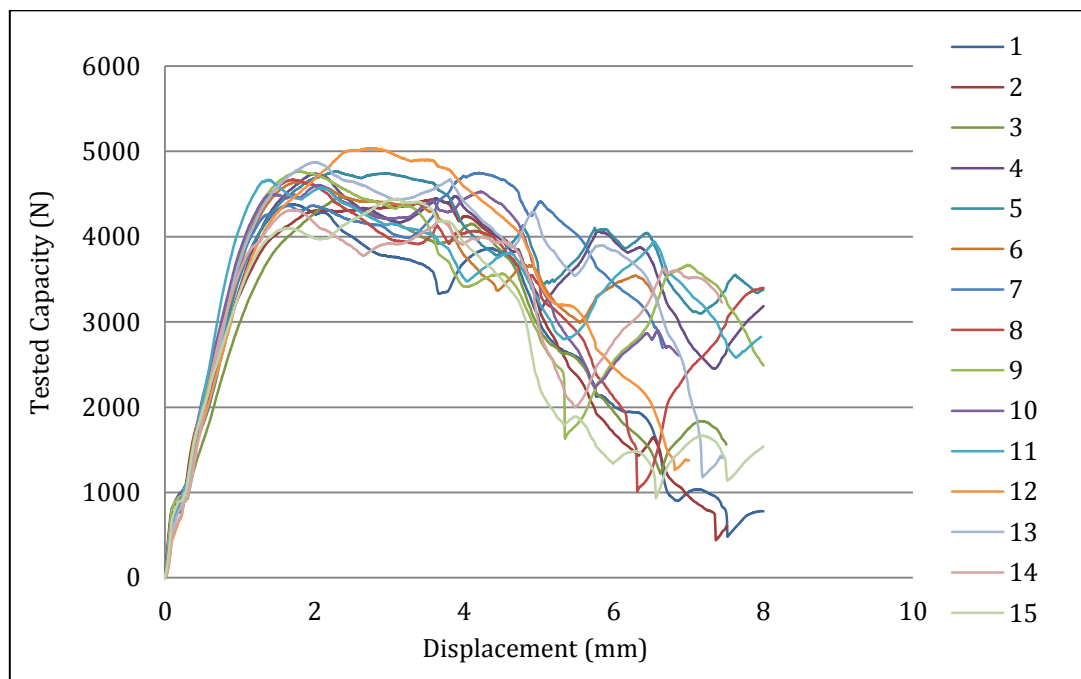


Figure A.24 Coil 3, Specimen 1-15

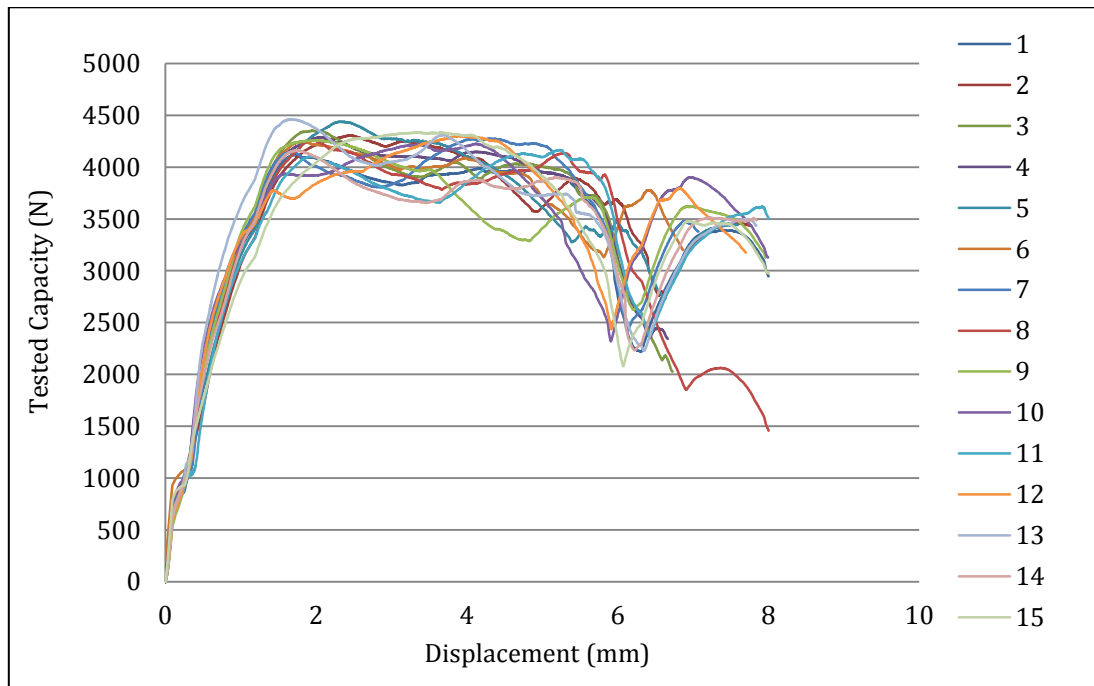


Figure A.25 Coil 4, Specimen 1-15

A.2.2 Typical connection 2 (Screwed, 0.58 mm, 550 MPa, 1 Coil, 30 Specimen)

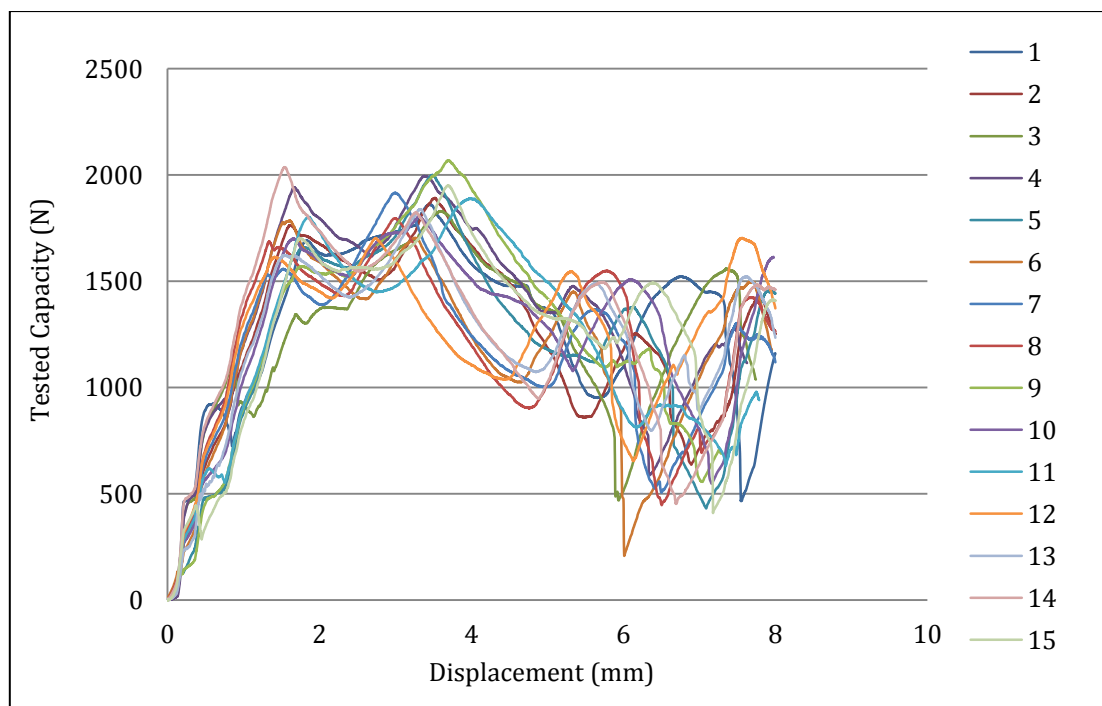


Figure A.26 Specimen 1-15

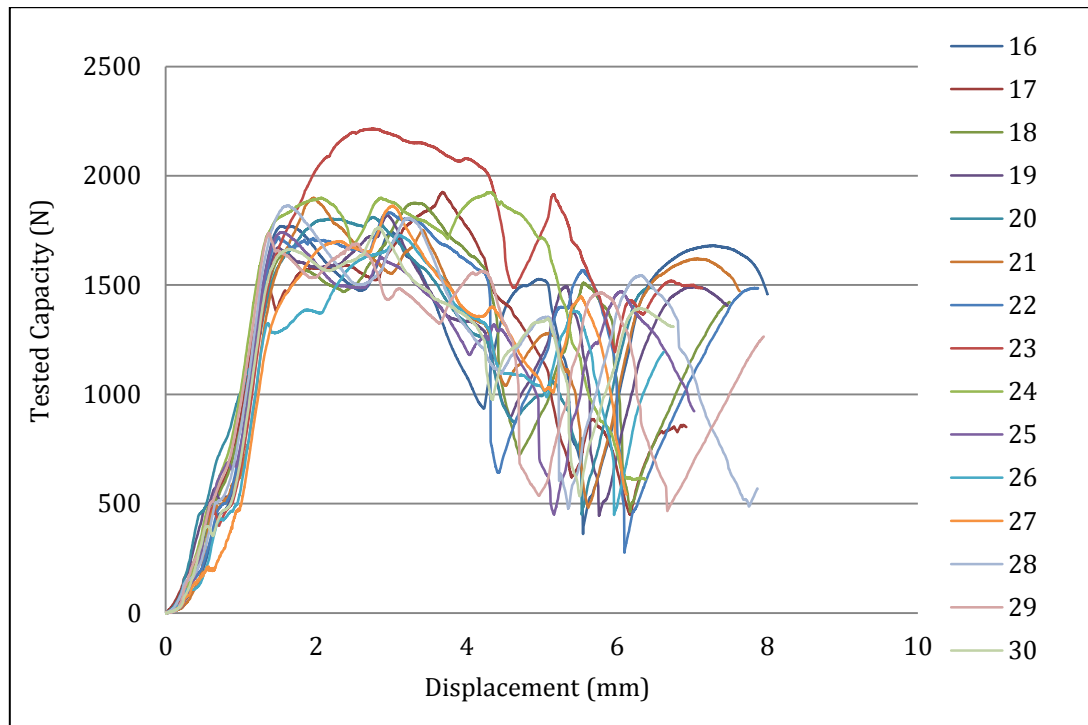


Figure A.27 Specimen 16-30

A.2.3 Typical connection 3 (Screwed, 1.0 mm, 300 MPa, 1 Coil, 30 Specimen)

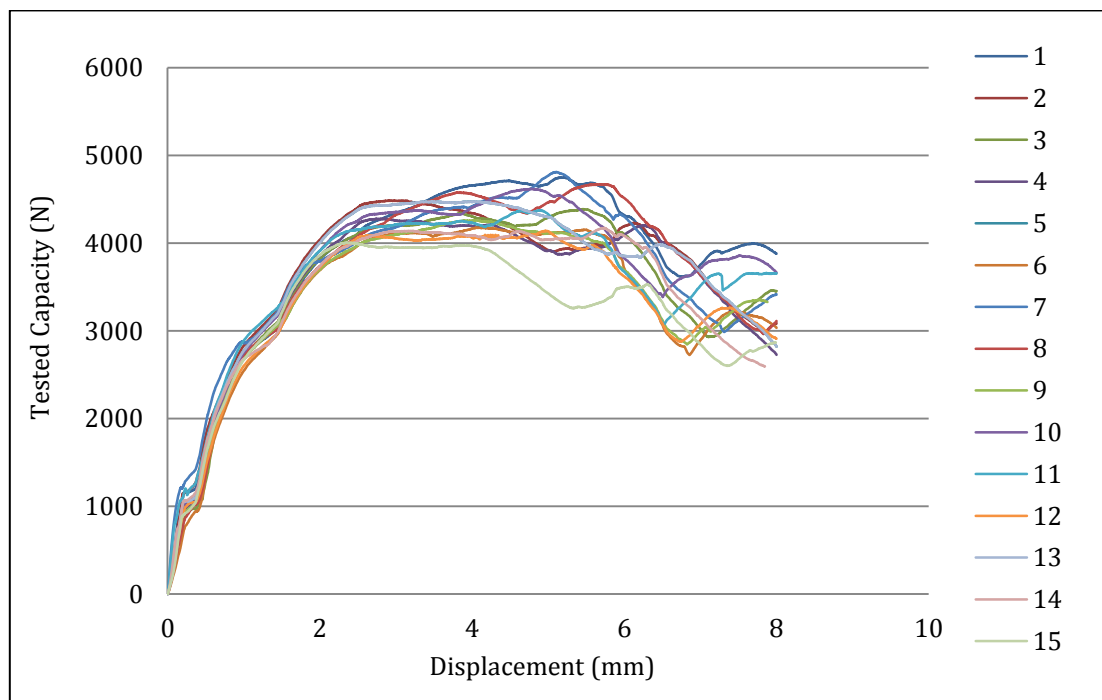


Figure A.28 Specimen 1-15

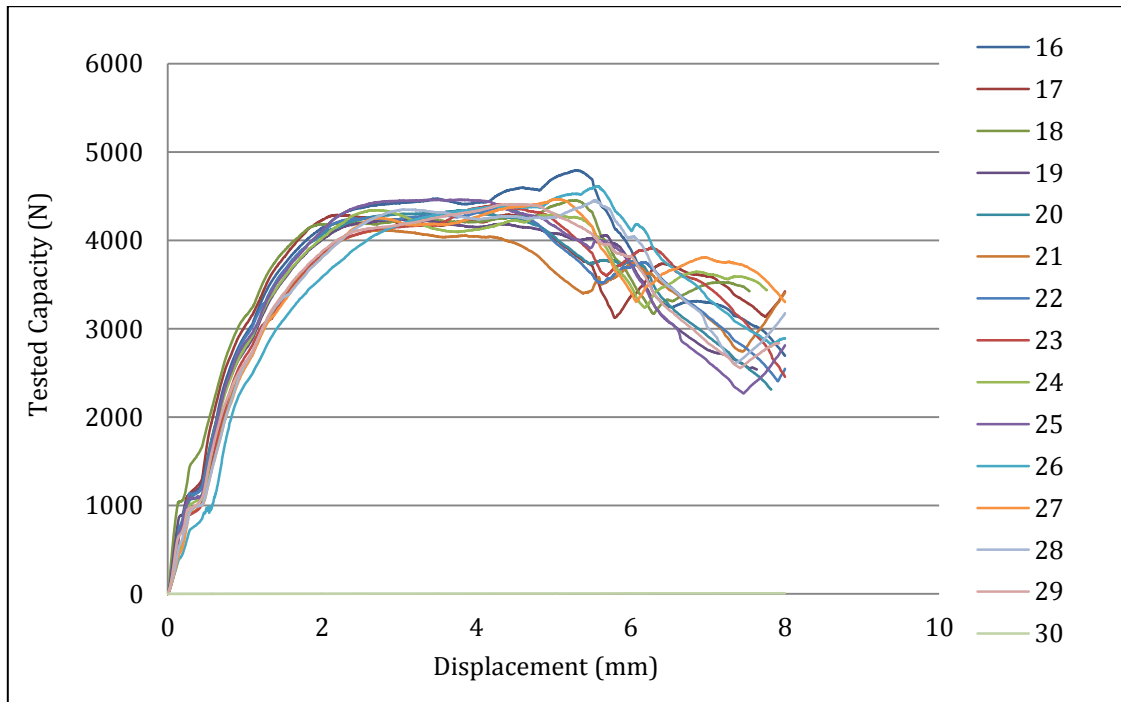


Figure A.29 Specimen 16-30

A.2.4 Typical connection 4 (Screwed, 0.8 mm, 230 MPa, 1 Coil, 30 Specimen)

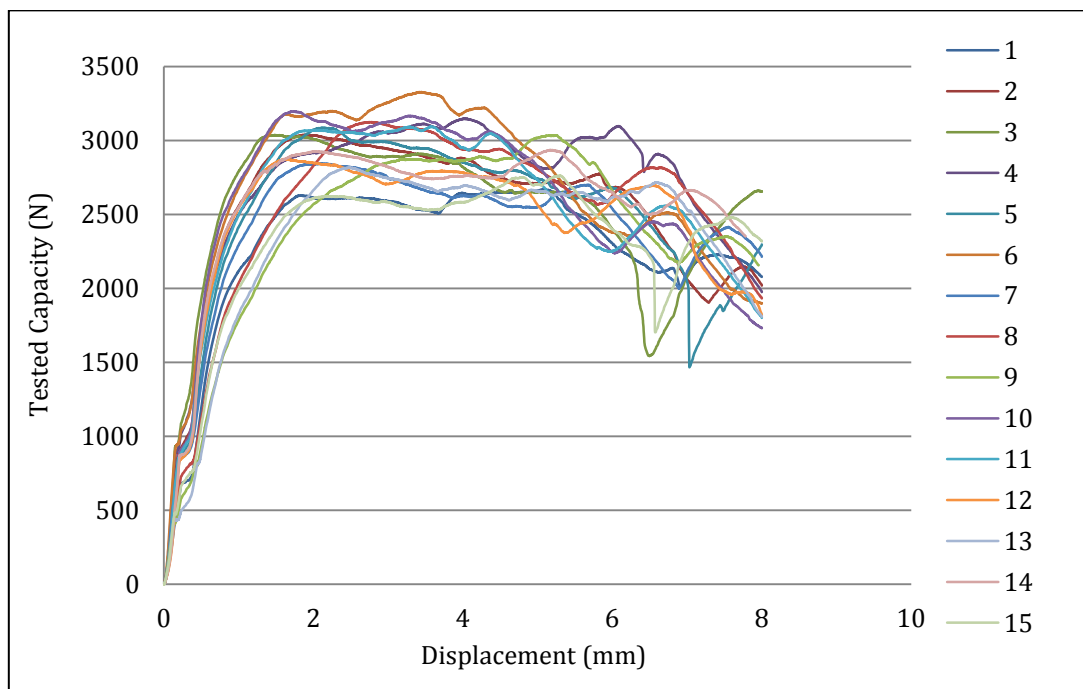


Figure A.30 Specimen 1-15

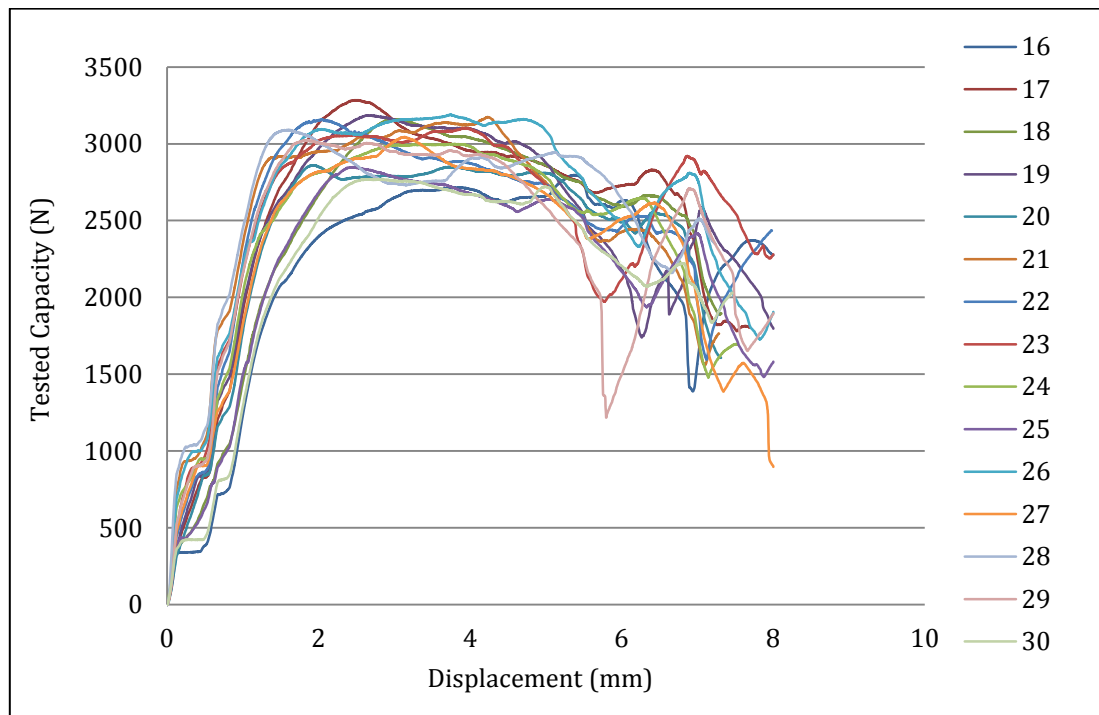


Figure A.31 Specimen 16-30

A.2.5 Typical connection 5 (Riveted, 0.8 mm, 550 MPa, 15 Specimen)

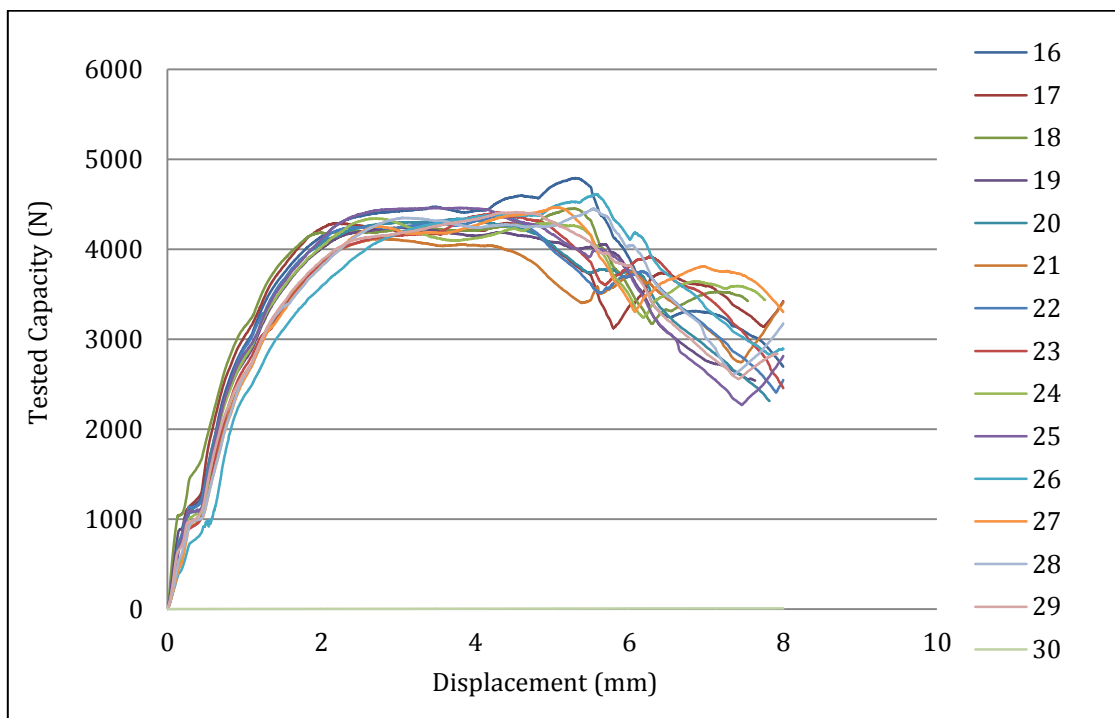


Figure A.32 Specimen 1-15



A.3 Model factor determination

Table A.4 Typical connection 1 (Screwed, 0.8 mm, 550 MPa, coil 1)

Specimen	Tested capacity (N)	Thickness (mm)	Diameter (mm)	Tensile strength (Mpa)	Predicted capacity (N)	Model factor (tested/predicted)
1	3926.66	0.76	4.73	605.76	3694.28	1.06
2	3285.32	0.77	4.73	605.76	3703.36	0.89
3	3281.50	0.77	4.75	605.76	3708.37	0.88
4	3830.19	0.77	4.71	605.76	3731.15	1.03
5	4041.42	0.77	4.81	605.76	3772.00	1.07
6	3815.31	0.77	4.75	605.76	3726.57	1.02
7	3794.91	0.77	4.87	605.76	3765.93	1.01
8	3893.51	0.77	4.80	605.76	3764.92	1.03
9	3927.51	0.77	4.84	605.76	3782.54	1.04
10	3700.98	0.77	4.72	605.76	3744.77	0.99
11	3743.91	0.77	4.83	605.76	3789.01	0.99
12	4066.92	0.78	4.72	605.76	3790.38	1.07
13	3695.88	0.77	4.71	605.76	3747.22	0.99
14	4057.57	0.77	4.83	605.76	3760.27	1.08
15	3770.26	0.77	4.69	605.76	3724.73	1.01
16	3712.03	0.77	4.77	605.76	3753.05	0.99
17	3788.54	0.78	4.86	605.76	3836.80	0.99
18	3742.21	0.77	4.82	605.76	3774.85	0.99
19	3732.01	0.77	4.69	605.76	3739.87	1.00
20	3975.11	0.77	4.76	605.76	3776.57	1.05
21	3551.38	0.77	4.80	605.76	3764.92	0.94
22	3812.76	0.77	4.76	605.76	3748.90	1.02
23	3831.46	0.77	4.78	605.76	3738.87	1.02
24	3969.16	0.77	4.85	605.76	3784.13	1.05
25	3908.81	0.77	4.77	605.76	3754.81	1.04
26	3831.46	0.77	4.74	605.76	3732.74	1.03
27	3797.89	0.77	4.75	605.76	3746.93	1.01
28	3953.44	0.77	4.80	605.76	3749.13	1.05
29	4014.22	0.77	4.80	605.76	3755.84	1.07
30	3516.53	0.77	4.78	605.76	3758.44	0.94
31	4025.69	0.77	4.86	605.76	3806.80	1.06
32	3753.26	0.78	4.79	605.76	3797.27	0.99
33	3838.69	0.77	4.75	605.76	3735.67	1.03
34	3694.18	0.77	4.79	605.76	3753.92	0.98
35	3817.44	0.77	4.82	605.76	3774.61	1.01
36	4230.12	0.77	4.73	605.76	3730.62	1.13
37	3583.68	0.77	4.75	605.76	3736.43	0.96
38	3897.34	0.77	4.75	605.76	3746.93	1.04
39	3916.04	0.77	4.85	605.76	3805.54	1.03
40	4334.39	0.78	4.71	605.76	3784.03	1.15
41	3770.12	0.77	4.77	605.76	3773.40	1.00



42	3937.53	0.77	4.75	605.76	3757.28	1.05
43	3794.34	0.77	4.76	605.76	3749.15	1.01
44	4080.30	0.77	4.77	605.76	3754.36	1.09
45	3891.64	0.77	4.76	606.76	3747.07	1.04
46	4188.22	0.77	4.82	605.76	3772.84	1.11
47	3904.39	0.77	4.79	605.76	3763.08	1.04
48	3729.33	0.77	4.85	605.76	3784.29	0.99
49	4076.48	0.77	4.77	605.76	3772.66	1.08
50	3902.27	0.77	4.78	605.76	3769.27	1.04
51	3998.72	0.77	4.79	605.76	3781.14	1.06
52	4066.70	0.77	4.77	605.76	3774.67	1.08
53	4265.98	0.77	4.70	605.76	3746.28	1.14
54	3885.05	0.77	4.79	605.76	3789.99	1.03
55	3956.85	0.77	4.82	605.76	3774.85	1.05
56	3888.02	0.77	4.79	605.76	3772.11	1.03
57	4172.26	0.77	4.75	605.76	3755.39	1.11
58	3885.47	0.77	4.75	605.76	3775.14	1.03
59	3971.72	0.78	4.82	605.76	3820.66	1.04
60	4026.96	0.77	4.83	605.76	3779.39	1.07
61	3803.90	0.77	4.79	605.76	3762.11	1.01
62	3923.29	0.77	4.85	605.76	3785.26	1.04
63	4034.60	0.77	4.68	605.76	3725.98	1.08
64	3803.47	0.77	4.75	605.76	3756.76	1.01
65	3940.28	0.77	4.72	605.76	3735.67	1.05
66	3930.51	0.77	4.77	605.76	3744.38	1.05
67	3925.41	0.77	4.76	605.76	3769.75	1.04
68	3902.04	0.77	4.80	605.76	3785.82	1.03
69	4291.65	0.78	4.76	605.76	3787.60	1.13
70	4097.91	0.77	4.70	605.76	3719.04	1.10
71	4105.98	0.77	4.82	605.76	3775.39	1.09
72	4054.15	0.77	4.79	605.76	3762.80	1.08
73	4043.63	0.77	4.80	605.76	3767.46	1.07
74	3818.87	0.77	4.82	605.76	3772.65	1.01
75	4042.78	0.77	4.83	605.76	3787.06	1.07
76	4108.64	0.77	4.85	605.76	3784.29	1.09
77	4103.12	0.77	4.79	605.76	3771.96	1.09
78	4175.77	0.77	4.81	605.76	3770.01	1.11
79	3928.49	0.77	4.70	605.76	3725.55	1.05
80	3842.66	0.77	4.74	605.76	3743.54	1.03
81	4157.93	0.77	4.84	605.76	3783.08	1.10
82	3765.33	0.77	4.80	605.76	3767.46	1.00
83	4122.24	0.77	4.82	605.76	3765.92	1.09
84	4020.26	0.77	4.82	605.76	3765.66	1.07
85	3932.31	0.78	4.94	605.76	3857.39	1.02
86	3876.23	0.78	4.80	605.76	3839.06	1.01
87	4209.76	0.77	4.81	605.76	3779.75	1.11



Table A.5 Typical connection 1 (Screwed, 0.8 mm, 550 MPa, coil 2)

Specimen	Tested capacity (N)	Thickness (mm)	Diameter (mm)	Tensile strength (Mpa)	Predicted capacity (N)	Model factor (tested/predicted)
1	4250.41	0.77	4.83	633.05	3946.53	1.08
2	4300.17	0.77	4.79	633.05	3930.95	1.09
3	4392.03	0.77	4.83	633.05	3967.92	1.11
4	4484.75	0.77	4.83	633.05	3947.00	1.14
5	4429.46	0.77	4.78	633.05	3927.51	1.13
6	4251.26	0.77	4.80	633.05	3934.34	1.08
7	4181.09	0.77	4.81	633.05	3938.21	1.06
8	4234.68	0.77	4.87	633.05	3955.53	1.07
9	4261.04	0.77	4.65	633.05	3890.80	1.10
10	4485.60	0.77	4.86	633.05	3960.44	1.13
11	4494.96	0.77	4.79	633.05	3933.28	1.14
12	4486.03	0.77	4.85	633.05	3954.73	1.13
13	4274.65	0.77	4.80	633.05	3944.67	1.08
14	4194.70	0.78	4.84	633.05	4000.80	1.05
15	4476.67	0.77	4.77	633.05	3902.68	1.15

Table A.6 Typical connection 1 (Screwed, 0.8 mm, 550 MPa, coil 3)

Specimen	Tested capacity (N)	Thickness (mm)	Diameter (mm)	Tensile strength (Mpa)	Predicted capacity (N)	Model factor (tested/predicted)
1	4378.11	0.76	4.80	761.80	4611.13	0.95
2	4445.25	0.75	4.87	761.80	4595.61	0.97
3	4460.13	0.76	4.89	761.80	4675.32	0.95
4	4737.64	0.76	4.82	761.80	4654.99	1.02
5	4764.41	0.76	4.83	761.80	4658.96	1.02
6	4643.29	0.75	4.70	761.80	4529.65	1.03
7	4747.41	0.76	4.81	761.80	4648.12	1.02
8	4667.09	0.76	4.80	761.80	4600.76	1.01
9	4765.69	0.76	4.84	761.80	4615.96	1.03
10	4605.05	0.76	4.82	761.80	4632.66	0.99
11	4664.12	0.76	4.78	761.80	4634.00	1.01
12	5035.97	0.76	4.77	761.80	4618.33	1.09
13	4873.63	0.76	4.80	761.80	4596.99	1.06
14	4311.39	0.75	4.77	761.80	4559.35	0.95
15	4445.25	0.76	4.82	761.80	4643.51	0.96

Table A.7 Typical connection 1 (Screwed, 0.8 mm, 550 MPa, coil 4)

Specimen	Tested capacity (N)	Thickness (mm)	Diameter (mm)	Tensile strength (Mpa)	Predicted capacity (N)	Model factor (tested/predicted)
1	4119.18	0.76	4.79	630.03	3845.60	1.07
2	4306.31	0.76	4.83	630.03	3843.63	1.12
3	4351.40	0.76	4.72	630.03	3836.14	1.13
4	4287.60	0.76	4.81	630.03	3863.73	1.11
5	4439.01	0.77	4.75	630.03	3889.22	1.14
6	4245.07	0.76	4.82	630.03	3847.73	1.10
7	4276.12	0.76	4.84	630.03	3857.13	1.11
8	4257.40	0.76	4.82	630.03	3850.92	1.11
9	4256.98	0.76	4.82	630.03	3877.17	1.10
10	4225.93	0.77	4.82	630.03	3886.69	1.09
11	4165.53	0.76	4.80	630.03	3841.33	1.08
12	4299.93	0.76	4.81	630.03	3846.96	1.12
13	4461.55	0.77	4.81	630.03	3921.71	1.14
14	4159.58	0.76	4.79	630.03	3829.14	1.09
15	4335.24	0.76	4.82	630.03	3841.43	1.13



Table A.8 Typical connection 2 (Screwed, 0.58 mm, 550 MPa, single coil)

Specimen	Tested capacity (N)	Thickness (mm)	Diameter (mm)	Tensile strength (Mpa)	Predicted capacity (N)	Model factor (tested/predicted)
1	1860.75	0.47	4.79	622.49	1859.13	1.00
2	1892.23	0.47	4.78	622.49	1864.51	1.01
3	1830.13	0.47	4.86	622.49	1857.21	0.99
4	1995.58	0.47	4.69	622.49	1831.53	1.09
5	2001.11	0.47	4.81	622.49	1854.77	1.08
6	1785.47	0.47	4.81	622.49	1846.97	0.97
7	1916.90	0.47	4.71	622.49	1835.55	1.04
8	1796.96	0.47	4.81	622.49	1848.14	0.97
9	2068.73	0.47	4.78	622.49	1864.07	1.11
10	1806.31	0.47	4.85	622.49	1862.75	0.97
11	1889.68	0.47	4.87	622.49	1859.16	1.02
12	1703.39	0.47	4.85	622.49	1854.49	0.92
13	1839.06	0.47	4.69	622.49	1824.16	1.01
14	2037.26	0.47	4.79	622.49	1851.21	1.10
15	1950.50	0.47	4.84	622.49	1852.71	1.05
16	1769.74	0.47	4.86	622.49	1856.29	0.95
17	1925.83	0.47	4.76	622.49	1837.82	1.05
18	1876.49	0.48	4.80	622.49	1874.49	1.00
19	1821.20	0.48	4.86	622.49	1886.84	0.97
20	1810.99	0.48	4.77	622.49	1891.27	0.96
21	1899.46	0.47	4.80	622.49	1868.03	1.02
22	1831.83	0.47	4.75	622.49	1858.89	0.99
23	2215.47	0.47	4.87	622.49	1858.25	1.19
24	1924.55	0.47	4.82	622.49	1850.09	1.04
25	1741.67	0.47	4.79	622.49	1859.18	0.94
26	1728.06	0.47	4.66	622.49	1817.86	0.95
27	1862.88	0.47	4.80	622.49	1845.59	1.01
28	1863.73	0.47	4.76	622.49	1852.16	1.01
29	1738.69	0.47	4.78	622.49	1820.30	0.96
30	1763.36	0.48	4.82	622.49	1879.93	0.94



Table A.9 Typical connection 3 (Screwed, 1.0 mm, 300 MPa, single coil)

Specimen	Tested capacity (N)	Thickness (mm)	Diameter (mm)	Tensile strength (Mpa)	Predicted capacity (N)	Model factor (tested/predicted)
1	4749.73	0.97	4.76	417.76	3663.99	1.30
2	4484.13	0.97	4.80	417.76	3671.14	1.22
3	4387.66	0.97	4.77	417.76	3653.63	1.20
4	4277.18	0.97	4.76	417.76	3671.07	1.17
5	4473.93	0.98	4.77	417.76	3690.13	1.21
6	4176.04	0.97	4.71	417.76	3659.15	1.14
7	4810.49	0.97	4.76	417.76	3671.07	1.31
8	4669.83	0.97	4.73	417.76	3646.47	1.28
9	4259.75	0.98	4.83	417.76	3741.82	1.14
10	4617.14	0.97	4.77	417.76	3675.67	1.26
11	4374.07	0.98	4.76	417.76	3683.91	1.19
13	4138.64	0.97	4.66	417.76	3591.05	1.16
14	4172.21	0.97	4.75	417.76	3638.51	1.10
15	4005.20	0.98	4.76	417.76	3700.93	1.29
16	4792.65	0.97	4.80	417.76	3665.88	1.17
17	4292.47	0.96	4.72	417.76	3587.36	1.24
18	4453.53	0.97	4.76	417.76	3677.46	1.15
19	4241.90	0.98	4.78	417.76	3721.05	1.17
20	4339.22	0.97	4.74	417.76	3671.18	1.12
21	4116.12	0.97	4.81	417.76	3663.23	1.19
22	4369.39	0.97	4.71	417.76	3631.39	1.21
23	4408.91	0.97	4.89	417.76	3697.93	1.17
24	4343.04	0.97	4.70	417.76	3639.63	1.23
25	4460.76	0.97	4.84	417.76	3679.17	1.25
26	4613.74	0.97	4.87	417.76	3699.01	1.21
27	4465.86	0.97	4.70	417.76	3642.44	1.22
28	4449.28	0.98	4.77	417.76	3687.58	1.20
29	4409.34	0.98	4.83	417.76	3728.12	1.19
30	4425.48	0.96	4.77	417.76	3620.45	1.22



Table A.10 Typical connection 2 (Screwed, 0.8 mm, 230 MPa, single coil)

Specimen	Tested capacity (N)	Thickness (mm)	Diameter (mm)	Tensile strength (Mpa)	Predicted capacity (N)	Model factor (tested/predicted)
1	2668.35	0.77	4.81	396.43	2449.52	1.09
2	3037.52	0.76	4.85	396.43	2447.58	1.24
3	3036.67	0.76	4.75	396.43	2404.65	1.26
4	3149.37	0.76	4.77	396.43	2420.65	1.30
5	3086.43	0.76	4.82	396.43	2433.58	1.27
6	3327.58	0.76	4.82	396.43	2438.99	1.36
7	2843.58	0.76	4.77	396.43	2427.09	1.17
8	3124.71	0.76	4.72	396.43	2407.94	1.30
9	3035.39	0.77	4.75	396.43	2427.98	1.25
10	3196.58	0.76	4.71	396.43	2394.32	1.34
11	3093.23	0.76	4.81	396.43	2419.68	1.28
12	2874.63	0.76	4.78	396.43	2411.45	1.19
13	2817.64	0.76	4.88	396.43	2437.92	1.16
14	2935.87	0.76	4.72	396.43	2396.84	1.22
15	2761.92	0.76	4.87	396.43	2445.82	1.13
16	2797.65	0.76	4.75	396.43	2397.88	1.17
17	3284.19	0.76	4.84	396.43	2407.85	1.36
18	3159.16	0.75	4.83	396.43	2370.24	1.33
19	3185.10	0.76	4.79	396.43	2403.39	1.33
20	2859.74	0.75	4.80	396.43	2381.97	1.20
21	3174.47	0.74	4.88	396.43	2330.16	1.36
22	3159.16	0.75	4.82	396.43	2361.56	1.34
23	3100.89	0.75	4.81	396.43	2378.46	1.30
24	2999.24	0.76	4.78	396.43	2424.25	1.24
25	2847.83	0.77	4.73	396.43	2428.31	1.17
26	3189.78	0.77	4.72	396.43	2438.18	1.31
27	3044.75	0.76	4.75	396.43	2423.31	1.26
28	3090.68	0.76	4.80	396.43	2417.98	1.28
29	3023.06	0.76	4.83	396.43	2419.38	1.25
30	2769.58	0.76	4.82	396.43	2422.70	1.14

Table A.11 Typical connection 2 (Riveted, 0.8 mm, 550 MPa, coil 4)

Specimen	Tested capacity (N)	Thickness (mm)	Diameter (mm)	Tensile strength (Mpa)	Predicted capacity (N)	Model factor (tested/predicted)
1	3239.99	0.76	4.80	630.03	3259.92	0.99
2	3314.42	0.76	4.82	630.03	3299.21	1.00
3	3474.33	0.76	4.82	630.03	3266.70	1.06
4	3464.55	0.76	4.82	630.03	3282.94	1.06
5	3309.74	0.76	4.84	630.03	3273.47	1.01
6	3428.83	0.76	4.79	630.03	3280.81	1.05
7	3382.47	0.75	4.80	630.03	3243.74	1.04
8	3333.13	0.76	4.80	630.03	3292.36	1.01
9	3524.94	0.76	4.85	630.03	3276.85	1.08
10	3437.33	0.76	4.79	630.03	3256.52	1.06
11	3136.64	0.75	4.82	630.03	3250.49	0.96
12	3171.94	0.76	4.82	630.03	3299.21	0.96
13	3069.44	0.76	4.80	630.03	3292.36	0.93
14	3151.95	0.76	4.80	630.03	3284.24	0.96
15	2978.85	0.76	4.80	630.03	3300.48	0.90



Appendix B

Statistical Parameters

The following selected statistical data is presented in this appendix:

Tensile testing (B.1)

- Average, standard deviation and coefficients of variation for:
 - Tensile test specimen cross sectional area.
 - Tensile forces.
 - Tensile stresses.

Connection testing (B.2)

- Average, standard deviation and coefficients of variation for:
 - Connected member thicknesses
 - Nominal fastener diameter
 - Model factor



B.1 Tensile test statistical parameters

Table B.1 Area and tensile force moment parameters

Steel property set (steel properties)	Area (mm ²)					Tensile Force (N)				
	A	Average μ	Sd σ	CoV (%)		F	Average μ	Sd σ	CoV (%)	
1 (0.8 mm, 550 MPa, Coil 1)	15.42	15.39	15.36	0.03	0.18	9363.85	9324.75	59.77	0.64	9.84
	15.43					9403.81				
	15.39					9276.29				
	15.36					9258.44				
	15.37					9363.85				
	15.39					9282.24				
1 (0.8 mm, 550 MPa, Coil 2)	15.54	15.53	15.36	0.02	0.12	9872.19	9832.22	56.53	967.87	0.57
	15.52					9792.24				
1 (0.8 mm, 550 MPa, Coil 3)	15.12	15.10	15.36	0.03	0.20	11579.65	11501.01	111.22	0.97	9.84
	15.08					11422.36				
1 (0.8 mm, 550 MPa, Coil 4)	15.37	15.37	15.36	0.01	0.07	9701.02	9680.82	28.57	0.30	9.84
	15.36					9660.62				
2 (0.58 mm, 550 MPa) (single coil)	9.52	9.47	9.47	0.07	0.77	5890.33	5892.46	3.00	0.05	0.05
	9.41					5894.58				
3 (1.0 mm, 300 MPa) (single coil)	19.25	19.08	19.08	0.14	0.75	8042.33	7971.73	90.53	1.14	1.14
	18.99					7869.67				
	19.01					8003.20				
	15.20					6023.75				
4 (0.8 mm, 230 MPa) (single coil)	15.11	15.08	15.08	0.11	0.75	6008.86	5976.86	48.65	0.81	0.81
	14.93					5917.85				
	15.07					5956.98				
	15.07					5956.98				

Table B.2 Yield and tensile stress moment parameters

Steel property set (steel properties)	Yield strength(Mpa)					Tensile strength(Mpa)				
	Fy	Average μ	Sd σ	CoV (%)		Fu	Average μ	Sd σ	CoV (%)	
1 (0.8 mm, 550 MPa, Coil 1)	604.88	603.46	603.46	4.36	0.72	607.25	605.76	3.28	0.54	10.72
	607.65					609.36				
	601.94					602.66				
	599.16					602.65				
	608.80					609.38				
	598.34					603.29				
1 (0.8 mm, 550 MPa, Coil 2)	631.01	630.44	630.44	0.80	0.13	635.08	633.05	2.87	70.49	0.45
	629.88					631.03				
1 (0.8 mm, 550 MPa, Coil 3)	765.18	760.92	760.92	6.02	0.79	765.91	761.80	5.82	0.76	0.76
	756.66					757.68				
1 (0.8 mm, 550 MPa, Coil 4)	630.23	629.63	629.63	0.85	0.13	631.03	630.03	1.42	0.22	0.22
	629.03					629.03				
2 (0.58 mm, 550 MPa) (single coil)	571.80	578.71	578.71	9.77	1.69	618.86	622.49	5.13	0.82	0.82
	585.63					626.11				
3 (1.0 mm, 300 MPa) (single coil)	338.87	338.50	338.50	0.90	0.27	417.87	417.76	3.37	0.81	0.81
	337.48					414.34				
	339.16					421.07				
4 (0.8 mm, 230 MPa) (single coil)	312.91	311.72	311.72	1.45	0.47	396.33	396.43	1.01	0.26	0.26
	312.56					397.71				
	311.74					396.44				
	309.68					395.23				



B.2 Connection test statistical parameters

Table B.3 Steel strip thickness moment parameters

Typical connection (steel properties)	Connector type	No. of Specimen	Sample size (No. of coils)	Original Thickness(mm)			Adjusted Thickness(mm)	
				Average μ	Std dev σ	CoV (%)	Average μ	Std dev σ
1 (0.8 mm, 550 MPa, Coil 1)	screwed	87	4	0.771	0.003	1	0.77	0.007
1 (0.8 mm, 550 MPa, Coil 2)	screwed	15		0.771	0.002			
1 (0.8 mm, 550 MPa, Coil 3)	screwed	15		0.757	0.003			
1 (0.8 mm, 550 MPa, Coil 4)	screwed	15		0.762	0.004			
2 (0.58 mm, 550 MPa, single coil)	screwed	30	1	0.472	0.002	0.51	0.55	0.007
3 (1.0 mm, 300 MPa, single coil)	screwed	30	1	0.971	0.005	0.47	0.97	0.007
4 (0.8 mm, 230 MPa, single coil)	screwed	30	1	0.759	0.006	0.84	0.77	0.007
5 (0.8 mm, 550 MPa, Coil 4)	riveted	15	1	0.757	0.003	0.39	0.77	0.007

Table B.4 Fastener diameter moment parameters

Typical connection (steel properties)	Connector type	No. of Specimen	Nominal diameter(mm)		
			Average μ	Std dev σ	CoV (%)
1 (0.8 mm, 550 MPa, Coil 1)	screwed	87	4.78	0.05	1.02
1 (0.8 mm, 550 MPa, Coil 2)	screwed	15	4.8	0.05	1.12
1 (0.8 mm, 550 MPa, Coil 3)	screwed	15	4.81	0.04	0.91
1 (0.8 mm, 550 MPa, Coil 4)	screwed	15	4.8	0.03	0.66
2 (0.58 mm, 550 MPa, single coil)	screwed	30	4.79	0.05	1.14
3 (1.0 mm, 300 MPa, single coil)	screwed	30	4.77	0.05	1.07
4 (0.8 mm, 230 MPa, single coil)	screwed	30	4.79	0.05	1.02
5 (0.8 mm, 550 MPa, Coil 4)	riveted	15	4.81	0.02	0.37
All screws in one set	screwed	222	4.79	0.05	1.03



Table B.5 Model factor moment parameters

Typical connection (steel properties)	Connector type	Model factor (Tested/Predicted)					
		Average μ		Std dev σ		CoV (%)	
1 (0.8 mm, 550 MPa, Coil 1)	screwed	1.04	1.05	0.05	0.05	4.73	5.19
1 (0.8 mm, 550 MPa, Coil 2)	screwed	1.1		0.03		2.92	
1 (0.8 mm, 550 MPa, Coil 3)	screwed	1		0.04		4.24	
1 (0.8 mm, 550 MPa, Coil 4)	screwed	1.11		0.02		1.91	
2 (0.58 mm, 550 MPa, single coil)	screwed	1.01		0.06		6.03	
3 (1.0 mm, 300 MPa, single coil)	screwed	1.2		0.05		4.38	
4 (0.8 mm, 230 MPa,single coil)	screwed	1.25		0.08		6.03	
5 (0.8 mm, 550 MPa, Coil 4)	riveted	1.01		0.05		5.19	